Chapter Six

DRAINAGE AND EROSION CONTROL

Surface runoff water is a serious threat to both the physical integrity and the serviceability of highway facilities. There is need to make certain that runoff water is adequately controlled so that it may pass through and be removed from the roadway area without damage to the roadway or adjacent properties.

Drainage design has two principal objectives: (1) to determine the amount of design storm runoff that can be expected at various points along the roadway, and (2) to determine the most appropriate type and size of facility that will effectively handle the estimated quantities of runoff. Guidelines and procedures are presented in this chapter for both of these objectives. The figures referred to in the text are presented at the end of the chapter.

General Policies and Criteria

Several basic policies, procedures and criteria have been adopted by the Department with regard to drainage design. These are described in the following paragraphs.

Design Responsibilities

Responsibilities for drainage design are divided between the Bridge Design Section and the Road Design Section -- based principally on the size of the drainage area involved and the type of structure to be provided.

The Bridge Design Section is responsible for:

- all locations requiring bridges,
- all concrete box culverts, and
- all drainage areas in excess of 300 acres.

The Road Design Section is responsible for:

- all locations with drainage areas up to 300 acres,
- storm sewer systems,
- open drain systems, and
- subsurface drainage.

Design of bank and channel protection should be performed by the unit responsible for design of the drainage structure.

Storm Design Frequency

The expected intensity of rainfall is a critical factor in drainage design. Normally, the intensity is measured in terms of inches per hour. Predicting the intensity of rainfall should be statistically based on actual recorded data over a considerable period of time.

Analysis of these data provides a reasonably realistic base for predicting the the maximum rainfall that might occur during design periods of various lengths of time. Statistically, the runoff resulting from a maximum storm during a 100-year, 30-minute storm will be greater than the runoff expected if only a 50-year, 30-minute storm is considered. For periods of 25 years or 10 years, the runoff from a maximum storm will probably be even less.

This approach gives an indication of what to expect. However, a 100-year storm might occur during the first 10 years of a design period. Another way of expressing the likelihood of a particular size storm is in terms of probability (expressed as a percentage) that the storm might occur in any one year. The probability that a 100-year storm may be exceeded is 1 percent for any given year. At the other extreme, there is a 10-percent probability that a 10-year storm may be exceeded in any given year.

The reasons for the concern about probability are economics and serviceability. A drainage structure designed for a 100-year storm will be much larger and more expensive than if it was designed for a 10-year storm. The extra cost of the larger structure must be compared with the potential damages resulting from occasionally exceeding the capacity of the smaller structure.

To establish uniformity and consistency, the Department has established several basic criteria for storm design frequency for particular situations. These are shown in Figure 6-8. These criteria represent guidelines for general application. Normally they should be used. However, unique local conditions or the permit requirements of other agencies may suggest use of a different design frequency at specific locations.

This is particularly true where the installation involves encroachment on an established flood plain. Such flood plains in Delaware are identified in a series of maps published by the Federal Emergency Management Agency (FEMA) for administering the National Flood Insurance Program. For projects within the established flood plains, designers shall always prepare a preliminary assessment of the potential effects of encroachments. Figure 6-9 shows an example form for documenting such assessments. Refer to "Procedures for Coordinating Encroachment on Flood Plains with FEMA."

Risk Analysis

When a preliminary assessment of local conditions indicates that alternative storm design frequencies should be considered, a formal risk analysis should be performed.

For example, a large and expensive drainage structure, designed to the prescribed storm design frequency, might be re-evaluated. What would be the consequences (both economic and environmental) of accepting a shorter design period and reducing the size of the structure? Is it worth the risk?

On the other hand, some areas adjacent to the highway facility may have a significant amount of critical and costly development. What risk is taken by using the adopted standard policy for the storm design period? Would it be justified to select a longer storm design period at this location, even though construction costs would be increased?

General policies and procedures for risk analysis are set forth in FHPM 6-7-3-2. A flow chart showing the procedures for compliance with waterway and flood plain management regulations is presented in Figure 6-10.

Minimum Sizes of Culverts

Department policy has established minimum sizes of culverts for particular installations -- even though these sizes may have capacity exceeding the design discharge requirements. The minimum permissible size for open-ended pipe culvert cross drains is 18". For all other pipe culvert installations for surface drainage (side drains, median drains, etc.) the minimum size is 15". The purpose of the minimum size is to facilitate pipe cleaning maintenance operations.

Documentation of Drainage Analysis

Designers must prepare drainage analyses for all projects and must maintain appropriate records of all drainage analyses performed. Standard forms have been prepared to facilitate the documentation. Examples are shown in following sections of this chapter. These forms, along with any supplementary worksheets and sketches, should be made a part of the permanent project file.

Surface Runoff Determination

A first step in drainage design is to estimate the surface runoff (or design discharge) at the location under consideration. Numerous factors influence this estimate, including (1) the rainfall intensity for a selected storm design frequency, (2) the size of the drainage area, and (3) the characteristics of the drainage area.

Two alternative estimating techniques have been adopted by the Department. The method to be used depends on the size of the drainage area.

1. The Rational Method is used for relatively small drainage areas (up to 300 acres).

2. The U.S.G.S. Method is used for larger drainage areas (over 300 acres).

Each of these two methods is described in the following sections.

Rational Method

The Rational Method uses an empirical formula relating rainfall to runoff. The formula is simple to use and is generally accepted as reliable for relatively small drainage areas. Because the formula assumes that the rainfall is of equal intensity over the entire drainage area, its use should be limited to drainage areas of up to about 300 acres.

The Rational Formula is given as:

$$Q = C\ I\ A$$

where Q = discharge in cfs;
C = runoff coefficient;
I = average rainfall intensity in inches per hour for the selected frequency; and
A = drainage area in acres.

The formula is based on the approximation that one inch of rainfall per hour per acre equals about one cubic foot per second, if $C = 1$. Thus, the calculation of Q is in terms of c.f.s.

The drainage area (A) is calculated in acres and usually can be measured from one or more of the following sources:

1. plans and actual field measurements,
2. interpretation of aerial photographs,
3. contour plans, and
4. U.S.G.S. maps.

The runoff coefficient (C) may be described as the percentage of the water falling on a given drainage basin that flows off directly. This value is expressed as a decimal and considers the following factors:

1. penetration and absorption by the soil and vegetation,
2. topography and slopes,
3. storage due to ponding, swamps, etc., and
4. evaporation.

These coefficients may vary from 0.10 (10%) in open sandy areas to almost 1.0 (100%) on paved surfaces. Values for C are shown in Figure 6-11. This table considers topography, vegetation and soil -- but does not consider storage due to ponding. The choice of a coefficient must be based on experience and judgment, using the table as a guide. The designer must also consider adjustments for possible changes in runoff characteristics due to land use.

The rainfall intensity (I) is dependent on the storm design frequency, the duration of the storm and the time of concentration. The time of concentration (Tc) is the overland flow time to the inlet plus the time in the drainage facility. The maximum flow at the point will occur when all points in the area are contributing runoff to the point at the same time. This maximum condition can occur only when the duration of the storm is equal to or exceeds the time of concentration. The time of concentration thus identifies the appropriate duration to be used in determining rainfall intensity.

The time of concentration for overland flow can be determined by either of two methods: (1) the nomograph shown in Figure 6-12, or (2) the equation shown below.

$$Tc = \left(60\sqrt[5]{\frac{A^2}{100S}}\right)\left(\frac{L}{5280\sqrt{\frac{4A}{\pi}}}\right)$$

where A = area in square miles;

S = slope in ft. per ft.;

L = length of drainage area in feet; and

Tc = time of concentration in minutes.

With the time of concentration established, the rainfall intensity can be determined from the appropriate Rainfall Intensity-Frequency Curves (Figures 6-13, 6-14, and 6-15).

When the values for C, I and A have been established, as described above, the design discharge is computed using the formula Q = C I A.

U.S.G.S. Method

For drainage areas of more than 300 acres, the U.S.G.S. Method should be used for determining the estimated surface runoff. Reference is made to the U.S.G.S. publication entitled "Techniques for Estimating Magnitude and Frequency of Floods in Delaware." The report was prepared in cooperation with the Delaware DOT and FHWA and was published in September 1978.

The technique was developed from extensive flood-discharge records at 60 gaging sites. From these data, equations were developed for estimating runoff at both gaged streams and ungaged streams. Separate equations are developed for northern and southern regions of the State, based on physical basin characteristics of each region.

It is not practical for this manual to attempt to reproduce all of the charts, tables, equations and explanations shown in the U.S.G.S. publication. Copies are available in the Office of Preconstruction Engineering. Designers should refer to and utilize the U.S.G.S. report for determining estimated surface runoff for the larger drainage areas.

Culvert Pipe Size Determination

After the design discharge has been identified, steps can be taken to determine the proper size of pipe culvert. The Department has adopted the procedures for pipe sizing set forth in Hydraulic Engineering Circular No. 5, "Hydraulic Charts for the Selection of Highway Culverts" (FHWA, 1965).

The publication is quite comprehensive, with detailed explanations of the hydraulic concepts and step-by-step procedures for determining pipe sizes at specific locations with consideration of variable conditions. Therefore, the information and charts are not duplicated in this manual. Reference is made to Hydraulic Engineering Circular No. 5 for pipe size determinations and to Hydraulic Engineering Circular No. 13 for improved inlets.

The procedures described above do not apply to sizing of pipe for storm drain systems. Practices for storm drains are discussed in a following section on storm drain design.

New Castle County Stormwater Management

It is pointed out that New Castle County has adopted special standards and criteria for Stormwater Management as documented in the publications "Stormwater Management" and "Specifications Guide for Sediment and Erosion Control," dated June 1977.

The purpose is to assure the application of planning, engineering and constructon principles to control the quantity and quality of storm runoff. Stormwater detention techniques (detention ponds, dry detention areas, rooftop ponding, parking-lot ponding, etc.) are prescribed to control the peak rate of surface runoff by temporary storage and controlled release of runoff.

Designers should refer to the described publications for drainage design guidance in New Castle County.

Storm Drain Design

Within the influence of urban areas, storm runoff waters collected on the roadway normally will be controlled with curb and gutter sections and will run off through storm sewer systems.

Municipal section drainage systems should limit the water ponding along the gutters and behind the curbs to amounts that will not interfere with traffic or damage property. This is accomplished by placing curb inlets, gutter inlets or drop inlets at specific locations so that water may enter the storm drain culverts. Also, inlets and underground drain systems normally are required in depressed medians of divided highways.

Guidelines and criteria for design of storm drain systems are set forth in the following sections.

Runoff Quantities

Because the drainage areas involved with storm drain systems usually are relatively small, the drainage runoffs normally should be determined by the Rational Method described previously in this chapter. The storm design frequency should be consistent with the values shown in Figure 6-8.

There is need for orderly tabulation of the runoff discharge computations to facilitate the storm sewer design. The form shown in Figure 6-3 (sample design problem) serves this purpose. Beginning in the uppermost reaches of the drainage system, tabulate by drainage area the amount of discharge accumulating along the sewer run. Work from the upper reaches down the system gradient to the point of outfall release.

Ponding Limits on Roadway

Water flow in gutters should be confined to a width and depth that will neither obstruct nor cause a hazard to traffic. The ponding width will have a corresponding depth, and the quantity of water carried will depend on the gutter gradient, gutter roughness coefficient, roadway cross section and inlet spacing.

The prescribed limits of ponding are:

- arterial highways or streets with full shoulder width or parking lane -- full shoulder width or parking lane width;
- arterial highways or streets with less than full shoulder width -- one half of adjacent lane;
- other multilane streets -- width of adjacent lane; and
- two-lane streets -- one third of adjacent lane.

The chart in Figure 6-16 is used for gutter capacity design. The chart is self-explanatory with instructions and examples. The ponding width (Zy) is set from the criteria above -- and the depth (y) is derived. With given restraints on ponding, determination can be made of the allowable accumulated runoff discharge that the gutter can accommodate; and this, in turn, will indicate the required spacing of inlets.

Inlet Spacing

Obviously, inlets must be placed at low points (or sags) in the gutterline. Additionally, they should be located at other points along grades where the accumulated runoff will approach the prescribed ponding limits.

Although the computed inlet spacing requirement may be of considerable distance -- possibly 500 to 600 feet -- actual spacing of inlets should be held to a maximum of about 300 feet, and often less, to facilitate pipe cleanout. In locations such as underpasses, and sag vertical curves in depressed sections, it is good engineering practice to place flanking inlets on each side of the inlet at the low point in the sag. For a more thorough discussion of this subject, refer to Hydraulic Engineering Circular No. 12 (March, 1984).

Pipe Sizing

Determination of required pipe sizes for storm drains is based on Manning's equation:

$$Q = \left[\frac{1.486}{n} \times R^{2/3} \times A\right] \times S^{1/2}$$

where Q = design discharge in cfs;

n = Manning's coefficient of roughness;

R = hydraulic radius, in feet;

A = pipe end area, in square feet; and

S = pipe slope, in feet per foot.

An important variable in the equation is "n," Manning's roughness coefficient. For reinforced concrete pipe, the coefficient is constant for all pipe sizes. For annular corrugated metal pipe, the coefficient is constant for all sizes -- but will vary if part or all of the inside of the pipe is paved. In the case of helical corrugated metal pipe, the coefficient varies with the size of pipe, and also with regard to the extent of paving. Figure 6-17 shows the appropriate values of "n" for the various types and sizes of pipe.

The value shown within the brackets in the Manning equation can be referred to as the "conveyance factor." The value for "Q" can be found by multiplying this factor by $S^{1/2}$. Figure 6-18 shows the conveyance factors for round corrugated pipe for various values of "n." Figure 6-19 shows the conveyance factors for corrugated arch pipe for various values of "n." And Figure 6-20 shows values for $S^{1/2}$ for various rates of slope.

Gradients

Storm drain grades should be established to assure a minimum velocity of at least 2 feet per second. The purpose is to prevent buildup of sediment.

Storm drain line gradients should be generally similar to the roadway grade. The same size of pipe will run until the cumulative discharge attains the pipe capacity. When an abrupt reduction in gradient is encountered, an increase of more than one pipe size larger may be required.

When increasing the size of pipe, two alternatives are available for design at the junction:

- Align the pipe inverts (inside bottom of pipe) with a continuous flow line; or

- Align the inside top of the pipe (soffit) with an abrupt drop in the flow line.

Each alternative has advantages and disadvantages. The hydraulic characteristics generally are better when the tops of the pipes are aligned. Also, this approach is better where there is a problem with minimum allowable cover over the pipes. On the other hand, there may be situations in relatively flat terrain where it is necessary to conserve the elevation of the flow line. Under these conditions it may be better to avoid the abrupt drops by aligning the pipe inverts at the junction.

Hydraulic Gradient

A hydraulic gradient is the line of elevations to which the water would rise in successive piezometer tubes along a storm drain run. Differences in elevations for the water surfaces in the successive tubes represent the energy loss for that length of storm drain.

The storm drain run will not be under pressure if it is placed on a calculated friction slope corresponding to a certain quantity of water, cross-section, and roughness factor -- and the surface of flow (hydraulic gradient) will be parallel to and below the top of the pipe. This is the desirable condition.

There may be reason to place the storm drain run on a slope less than the friction slope. In that case, the hydraulic gradient would be steeper than the slope of the storm drain run. Depending on the elevation of the hydraulic gradient at the downstream end of the run, it is possible to have the hydraulic gradient rise above the top of the pipe, creating pressure on the storm drain system until the hydraulic gradient at some point upstream is once again at or below the top of the pipe.

The hydraulic gradient is determined starting at the downstream end of the proposed system. Where the system is connected to all existing drainage systems, the hydraulic gradient at the point of junction shall be determined from the hydraulic gradient computations for the existing drain. If the proposed system is to discharge into a stream, flow conditions of the stream shall be investigated. Where the tailwater elevation is higher than the proposed crown elevation, the hydraulic gradient will begin at this tailwater elevation. If free outfall conditions exist, the gradient will begin at the crown of the proposed drain.

Next, the friction loss in the pipe to the next structure is added to the gradient. Then the head loss in the structure is added. The hydraulic gradient to the upstream end is thus determined by adding a series of friction losses in pipes and head losses in structures. To avoid creating a pressurized system (major cause of blow ups and joint failures), the hydraulic gradient should not exceed the pipe crown.

Head loss in pipes due to friction can be calculated using Manning's formula, by solving for S_f.

$$S_f = \left(\frac{Qn}{1.486\ AR^{2/3}}\right)^2$$

where Q = discharge (cfs);

n = roughness coefficient;

A = cross sectional area (sq. ft.); and

R = hydraulic radius (D/4 for circular pipes) (ft.).

The head loss is calculated from the formula:

$$H_f = S_f \times L$$

where L = length of pipe (ft.)

To compute head losses in structures, Figures 6-21 and 6-22 should be used. These curves were prepared for the determination of head loss in cut-ins, wye branches, preformed concrete pipe fittings, manholes, brick bends, and Type I junction chambers.

There are four curves, designated as "A," "B," "C," and "D" losses. The "A" curve gives losses due to entrance and exit. The "B" curve depicts velocity head. When there is a change in velocity, the difference in the velocity heads ($V_{H-2} - V_{H-1}$) is the head loss. If the upstream velocity is greater, this difference will be negative and the apparent gain may be used to offset other losses in the structure. The "C" curve shows losses in a manhole due to change in direction, loss in wye branch, and loss in brick bend. The "D" curve depicts losses due to entrance of secondary flows into a structure.

Computation of the hydraulic grade line will not be necessary where the following conditions are satisfied:

1. The slope and the pipe sizes are chosen so that the slope is equal to or greater than the friction slope;

2. The top surfaces of successive pipes are aligned at changes in size (rather than flow lines being aligned); and

3. The surface of the tailwater at the point of discharge does not rise above the top of the outlet.

The pipe will not operate under pressure in these cases, and the slope of the water surface under capacity discharge will approximately parallel the slope of the pipe invert. Small head losses at inlets, manholes, etc., may be disregarded if these structures are properly designed.

However, in cases where different sized pipe inverts are placed on the same grade, causing the smaller pipe to discharge against head, or when it is desired to check the storm drain system against larger-than-design floods, it will be necessary to compute the hydraulic grade of the entire storm system. Begin with the tailwater elevation at the storm drain outfall and progress upward the length of the storm drain. For every run, compute the friction loss and plot the elevation of the total head at each manhole and inlet.

If the hydraulic grade line rises above the top of any manhole or above an inlet entrance, the storm drain system is unsatisfactory because blowouts will occur through manholes and inlets. Pipe sizes or gradients must be increased as necessary to eliminate such blowouts.

A hydraulic gradient must have an original base elevation above the outlet tailwater elevation. Any backwater effects due to a significant tailwater elevation must be considered carefully.

Typical details for drop inlets, manholes and grates are shown in the Department's Standard Sheets.

Conflict with Underground Utilities

Construction of new storm drains raises the possibility of conflicts with existing underground utilities. Designers should check thoroughly for underground utilities before commencing design. Often it is possible to avoid conflicts by making minor adjustments in the line or grade of the storm drain. All recognized conflicts should be clearly identified and brought to the attention of the Utilities Section.

Hydraulic Analysis of Inlet Grates

The flow intercepted by an inlet grate consists of two parts: (1) frontal flow, the portion of the flow which passes over the upstream edge of the grate, and (2) side flow, which passes over the edge of the grate parallel to and away from the curb. The percent of frontal flow intercepted depends on the bar configuration, grate length, and flow velocity. All of the frontal flow will normally be intercepted on mild slopes. On steep slopes, the water may splash over the grate and not be intercepted. The amount of side flow intercepted decreases with increasing velocity, and increases with increasing grate length.

The hydraulic efficiency (E) of a grate is defined as the ratio of the total flow intercepted (Q_i) to the total gutter flow (Q_T).

$$E = Q_i/Q_T$$

For grates on a continuous slope, the quantity of flow intercepted increases as the flow rate increases. Thus, a percentage of the gutter flow may be allowed to flow around the inlet, to be picked up by downstream inlets or at the sump. The spacing of inlets on continuous grades is therefore determined by the allowable width of water on the pavement and the efficiency of the inlets.

If it is assumed that there is no side flow interception (only the flow passing over the front edge enters the grate) then the ratio of approach frontal flow (Q_F) to total gutter flow (Q_T) is given by the equation:

$$\frac{Q_F}{Q_T} = 1 - \left[1 - \frac{W}{T}\right]^{8/3}$$

where W = width of grate (ft.); and

T = spread on pavement (ft.).

To account for side flow, an effective width, W_E, is substituted for W in the above equation. The effective width is equal to the actual width plus the extra width, ΔW, that would be necessary for the inlet to have the same efficiency without side flow interception. ΔW is a function of longitudinal slope, cross slope, grate size, and bar configuration. Values for ΔW for eight grate configurations can be obtained from from charts in Chapter 5 of the FHWA publication "Design of Urban Highway Drainage -- The State of the Art" (August 1979).

The equation for calculating the grate inlet efficiency is then:

$$E_o = 1 - \left[1 - \frac{W_E}{T}\right]^{8/3}$$

This relationship applies only for no-splash conditions. If splashing will occur, this equation must be multiplied by a reduction factor, R. (See referenced FHWA publication.) The hydraulic efficiency is related to the inlet efficiency by the equation:

$$E = RE_o$$

Grate inlets should be designed longer than necessary for 100% frontal flow interception to allow for debris accumulation and clogging. It is recommended that grates be designed with a factor of safety of at least 1.5. The factor of safety is defined for a particular effective grate length, L', as the ratio of the frontal velocity at which 100% of the frontal flow is intercepted to the actual frontal velocity.

For a more thorough discussion of this subject, refer to Hydraulic Engineering Circular No. 12 (March, 1984).

Summary of Design Steps

The steps for designing storm-drainage system are as follows.

1. Prepare a drainage-area map using:

   a. U.S.G.S. maps and contour maps;

b. existing drainage structures and their elevations (an accurate field investigation must be performed);

c. field data -- outlet and scour conditions, ground surface type and proposed land use of drainage area -- and

d. soil type and water table elevations, which are needed for subsurface drainage.

2. Divide drainage area into subareas tributary to the proposed storm inlets, showing drainage limits, streets, impervious areas and direction of flow.

3. Compute acreage of each subarea.

4. Determine the appropriate runoff coefficient (C) for each subarea.

5. Determine the appropriate storm frequency to be used in design (Figures 6-13 through 6-15).

6. Determine the initial time of concentration (Tc) from Figure 6-12 (Tc_{min} = 5 Min.), and the rainfall intensity for this Tc from the appropriate rainfall intensity-frequency curve.

7. Using existing survey and field data, lay out a tentative drainage system, giving lengths and slopes of pipes, type and number of catch basins or manholes, and the direction of flow. Begin the profile at the point farthest downstream, which can be an outfall into a natural or artificial channel, or into an existing drain. Pipe slopes should conform to the surface slope wherever possible.

8. Complete the storm drainage design table using the proposed lengths and slopes to determine the proper pipe sizes necessary to accommodate the estimated runoff. (Detailed instructions for the completion of the table are given in subsequent sections.)

9. Plot profile of the proposed system using the pipe sizes calculated in Step 8. At all drainage structures, indicate the necessary change in invert elevations. Where there is no change in pipe size through a structure, a drop of 0.2 feet should be used if fall is available. Where the size increases downstream through a structure, the inside top of the pipes should be aligned, providing an invert drop at the outlet equal to the difference in the two pipe diameters.

10. Determine the hydraulic gradient for the system by calculating the friction head for each reach and the head loss in each structure. To avoid blowouts, it is desirable that the gradient not come within 1.5 feet of the surface. The system must be modified to eliminate this condition if it exists. This could be accomplished by reducing head losses, increasing the depth of the structure, or both, depending upon cost.

11. Analyze the curb, gutter and inlet hydraulics for the proposed types, to determine their capacities, flow depth, and spread. Also compute the probable depths at structure inlets. The ponding and spreading of flow must not exceed the limits specified. The nomograph, Figure 6-16, can be used to determine flow depths and spread in gutters. For grate inlets in sumps, Figure 6-23 is useful, and refer to HEC No. 12 (March, 1984).

12. Using sound engineering judgment, and taking into consideration the pipe sizes needed (Step 8), the hydraulic gradient (Step 10) and the actual quantity of water that gets into the system (Step 11), complete the design for the drainage system.

Use of Storm Drainage Design Sheet

The Storm Drainage Design Sheet (see Figure 6-3) is the basic documentation of the design process. The columns should be completed as follows.

1. Location: Manholes and inlets are to be numbered. The corresponding numbers for each reach are then entered in the first columns, starting at the upstream end of the system.

2. Increment area: Enter the acreage of the tributary areas for each reach.

3. Initial time of concentration: Determine from the overland flow time chart (Figure 6-12) plus the time in the facility.

4. Rainfall intensity: Determine from the appropriate intensity-frequency curve.

5. Runoff coefficient: Some tributary areas may contain portions that have different permeabilities (e.g., part grass, part pavement.). An average runoff coefficient must be calculated. This is done by calculating the area of each portion of the sub-area having a different coefficient. The average coefficient is then the sum of the products of these areas and their respective coefficients, divided by the total area.

6. Discharge: Beginning at the upstream end of the system, compute the discharge to be carried by each successive length of pipe. Note that at each point downstream where a new flow is introduced, a new time of concentration must be determined.

7. Pipe size: Select the appropriate slope and use Figures 6-18, 6-19 and 6-20 to determine the pipe size.

8. Just full capacity and mean velocity: Determine the capacity of the pipe when it is flowing full from the figures referenced in Step 7. Divide this capacity by the cross-sectional area of the pipe to obtain the mean velocity.

9. Flow time: First compute the actual velocity of the flow in a pipe from the ratio of the actual discharge to the just full capacity (Q_{ACT}/Q_{JF}) and the mean velocity using Figure 6-24 or 6-25. Divide

the length of the pipe by this velocity to obtain the time of flow through the pipe.

Sample Design Problem

The following example problem is shown to illustrate the design steps, the use of the various charts and nomographs, and the preparation of the Storm Drainage Design Sheets.

The example involves a roadway that crosses a small valley at Station 205+95 with -1.30% and +1.83% grades resulting in a sump at the center of a 200-foot vertical curve. As Figure 6-1 shows, grated inlets catch the runoff in curbed gutters at Station 204+00 and at the gutter sumps at Station 205+78.1. In addition, sodded ditches intercept the runoff from the drainage area to the south of the highway, and these ditch runoffs are collected by grated inlets in the ditch at 204+00 and at the low sag at 205+95. The runoff from the inlets on the south edge of the highway is then conveyed under the highway where the north-side inlets are picked up and the accumulated runoff is discharged into a small natural watercourse.

The traversed area is suburban in character, and its zoning indicates residential apartments assumed to result in 70% imperviousness.

The rational method is to be used and the runoff coefficients for pervious and impervious areas are assumed to be 0.30 and 0.95, respectively. The system will be designed for a 10-year storm.

Design Steps

1-4. The runoff coefficients and acreage of the tributary areas for each inlet are summarized in Figure 6-3.

5. The storm frequency to be used in design is 10 years.

6. The initial time of concentration is determined for inlet 1 from the overland flowtime nomograph (Figure 6-12). For this case the length of the strip (i.e., the distance between the inlet and the point farthest from it within its tributary area) is 500'. The ground is an average grass surface, and the slope of the ground is 1%. For these conditions, Tc = 26.5 min. The rainfall intensity for this Tc in Sussex County (see Figure 6-15) is 3.85 in/hr.

7. The selected tentative pipe sizes and slopes are shown on the location and profile sheets as well as the storm drainage design sheet.

8. Figure 6-3 is the completed Storm Drainage Design sheet for this example.

9. Figure 6-2 shows the profile for the proposed system.

10. <u>Computing the Hydraulic Gradient</u>

    The losses in each structure and reach are computed as follows and documented as shown in Figure 6-4:

<u>INLET 1</u>

A. Velocity diagram: $V_R \dashrightarrow$ ① $\xrightarrow[V_2]{}$ ②

V_R = 0 (due to overland flow)
V_2 = 7.99 fps exit velocity

B. Flow diagram: $Q_R \dashrightarrow$ ① $\xrightarrow[Q^2]{}$ ②

$Q_R = Q_2$ = 7.53 cfs Initial system inlet

C. Head loss in structures (Figures 6-21 and 6-22)

1. Entrance head loss from V_1 to V_2

   From Curve "A" H_L = <u>0.35 ft.</u>

2. Head loss due to change in velocity from V_R to V_2

   From Curve "B" $H_{L2} - H_{LR}$ = <u>0.99 ft.</u>

3. Head loss due to bends:

   $H_L = \underline{0}$

4. Head loss due to secondary flows:

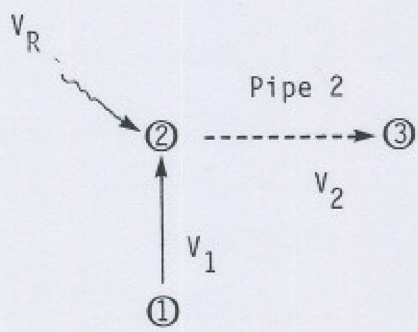
   $H_L = \underline{0}$

   Total H_L = <u>1.34 ft.</u>

INLET 2

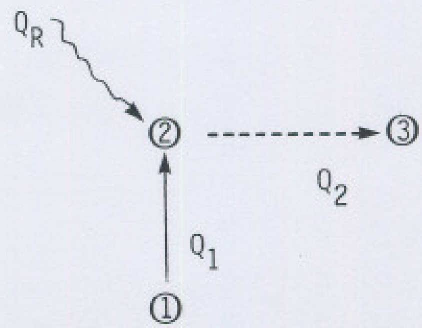
A. Velocity diagram:

$V_R = 0$
$V_1 = 7.99$ fps
$V_2 = 7.53$ fps



B. Flow diagram:

$Q_R = 1.40$ cfs
$Q_1 = 7.53$ cfs
$Q_2 = Q_R + Q_1 = 8.93$ cfs



C. Head loss in structures:

1. Entrance head loss -- using largest entrance velocity (V_1).

   From Curve "A" H_L = <u>0.35 ft.</u>

2. Head loss due to change in velocity from V_1 to V_2.

   From Curve "B" $H_{L2} - H_{L1} = 0.90 - 0.99 =$ <u>-0.9 ft.</u>

3. Head loss due to bends (90^o bend)

   From Curve "C" using largest existing velocity

   $H_L = 0.19 \times 2.0 =$ <u>0.38 ft.</u>

4. Head loss due to secondary flows. (Figure 6-22)

   From Curve "D" (Q_R = secondary flow, Q_1 = upstream flow)

   $$\frac{Q_R}{Q_1} = \frac{1.40 \text{ cfs}}{7.53 \text{ cfs}} = 0.19 = 19\%$$

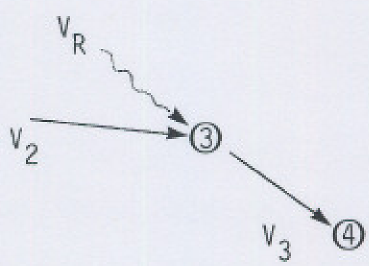
   Using $V_1 = 7.99$ fps H_L = <u>0.17 ft.</u>

   Total $H_L = 0.81$ ft.

<u>INLET 3</u>

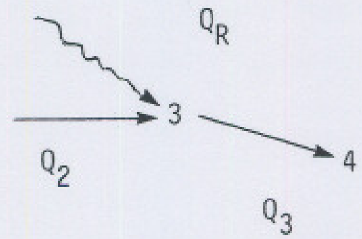
A. Velocity diagram:

$V_R = 0$
$V_2 = 7.53$ fps
$V_3 = 9.25$ fps



B. Flow diagram:

Q_R = 1.71 cfs
Q_2 = 8.93 cfs
$Q_3 = Q_R + Q_2$ = 10.64 cfs



C. Head loss in structures:

1. Entrance head loss -- using largest entrance velocity (V_2).

   H_L = <u>0.35 ft.</u>

2. Head loss due to change in velocity from V_2 to V_3.

   From Curve "B" $H_{L3} - H_{L2} = 1.31 - 0.91 =$ <u>0.40 ft.</u>

3. Head loss due to bends (17^o bend)

   From Curve "C" using largest existing velocity (V_3)

   $H_L = 0.25 \times 1/3(.25) =$ <u>0.17 ft.</u>

4. Head loss due to secondary flows

   From Curve "D" (Q_R = 1.71 cfs secondary flow, Q_2 = 8.93 cfs upstream flow)

   $$\frac{Q_R}{Q_2} = \frac{1.71 \text{ cfs}}{8.93 \text{ cfs}} = 0.19 = 19\%$$

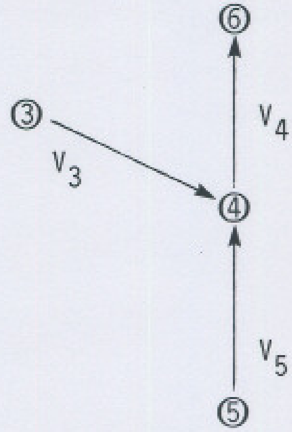
   Using V_2 = 7.53 fps H_L = <u>0.15 ft.</u>

   Total H_L = 1.07 fts.

INLET 4

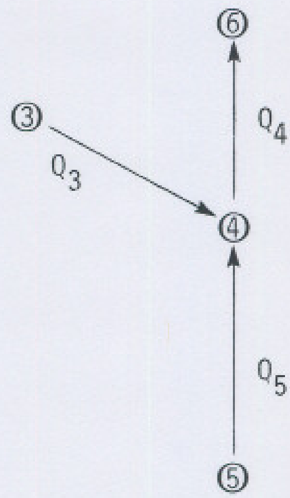
A. Velocity diagram:

V_3 = 9.25 fs
V_4 = 6.50 fs
V_5 = 5.08 fs



B. Flow diagram:

Q_3 = 10.64 cfs
Q4 = $Q_3 + Q_5$ = 13.04 cfs
Q5 = 2.40 cfs



C. Head loss in structures:

1. Entrance head loss -- using largest entrance velocity (V_3).

   From Curve "A" H_L = <u>0.43 ft.</u>

2. Head loss due to change in velocity from V_3 to V_4.

   From Curve "B" $H_{L4} - H_{L3}$ = .67 - 1.36 = <u>0.69 ft.</u>

3. Head loss due to bends (90^o bend)

   From Curve "C" using largest existing velocity
   H_L = 0.25 x 2.0 = <u>0.50 ft.</u>

4. Head loss due to secondary flows

   From Curve "D" (Q_5 = 2.40 cfs secondary flow, Q_1 = 9.25 cfs upstream flow)

   $$\frac{Q_5}{Q_1} = \frac{2.40\ \text{cfs}}{9.25\ \text{cfs}} = 0.26 = 26\%$$

   Using V_3 = 9.25 cfs H_L = <u>0.33 ft.</u>

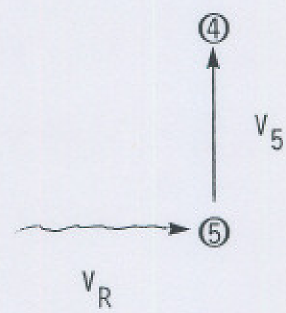
   Total H_L = <u>0.57 ft.</u>

<u>**INLET 5**</u>

A. Velocity diagram:

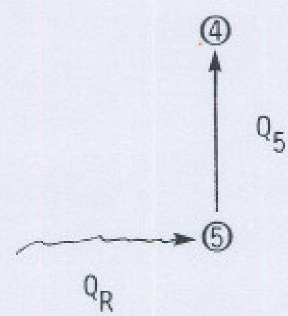
   $V_R = 0$

   $V_5 = 5.08$ fs



B. Flow diagram:

   $Q_R = Q_5 = 2.40$ cfs



C. Head loss in structures:

   1. Entrance head loss -- using largest entrance velocity (V_5).

      H_L = <u>0.15 ft.</u>

   2. Head loss due to change in velocity from V_R to V_5.

      From Curve "B" $H_{L5} - H_{L6} = (0.40 - 0)$ = <u>0.40 ft.</u>

3. Head loss due to bends (0 bend)

$H_L = 0$

4. Head loss due to secondary flows

No secondary

$H_L = 0$

Total $H_L = 0.55$ ft.

INLET 7

A. Velocity diagram:

$V_R = 0$
$V_7 = 4.14$ cfs

$$V_R \longrightarrow ⑦ \xrightarrow[V_7]{} ⑥$$

B. Flow diagram:

$Q_R = Q_7 = 1.86$ cfs

$$Q_R \longrightarrow ⑦ \xrightarrow[Q_7]{} ⑥$$

C. Head loss in structures:

1. Entrance head loss -- using largest entrance velocity (V_7).

From Curve "A" H_L = <u>0.11 ft.</u>

2. Head loss due to change in velocity from V_R to V_7.

From Curve "B" $H_{L7} - H_{LR} = (0.27 - 0) =$ <u>0.27 ft.</u>

3. Head loss due to bends (0 bend)

$H_L = 0$

4. Head loss due to secondary flows

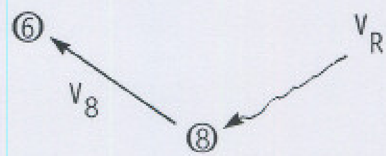
No secondary

$H_L = 0$

Total $H_L = 0.38$ ft.

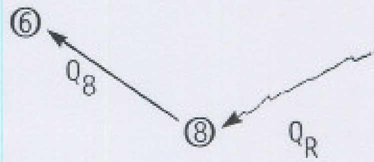
INLET 8

A. Velocity diagram:



$V_R = 0$
$V_8 = 5.20$ cfs

B. Flow diagram:



$Q_R = Q_8 = 1.68$

C. Head loss in structures:

1. Entrance head loss -- using largest entrance velocity (V_8).

   From Curve "A" H_L = <u>0.16 ft.</u>

2. Head loss due to change in velocity from V_8 to V_R.

   From Curve "B" $H_{L8} - H_{LR} = (0.43 - 0) =$ <u>0.43 ft.</u>

3. Head loss due to bends (0 bend)

   $H_L = 0$

4. Head loss due to secondary flows
   No secondary

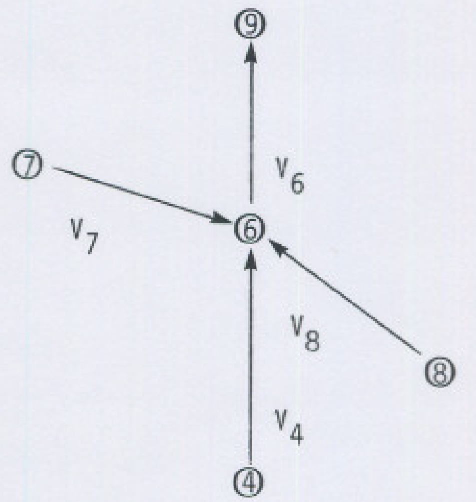
   $H_L = 0$

   Total H_L = <u>0.59 ft.</u>

INLET 6

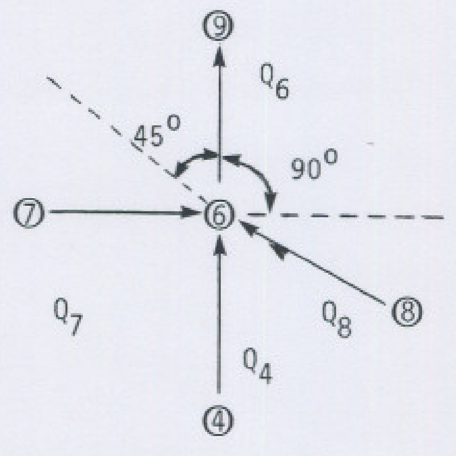
A. Velocity diagram:

$V_4 = 6.50$ fps
$V_6 = 6.10$ fps
$V_7 = 4.14$ fps
$V_8 = 5.20$ fps



B. Flow diagram:

Q4 = 13.04 cfs
Q6 = 16.58 cfs
Q7 = 1.86 cfs
Q8 = 1.68 cfs



C. Head loss in structures:

1. Entrance head loss -- using largest entrance velocity (V_4).

   From Curve "A" H_L = <u>0.23 ft.</u>

2. Head loss due to change in velocity from V_4 to V_6.

From Curve "B" $H_{L6} - H_{L4} = (0.60 - 0.66) = \underline{0.06 \text{ ft.}}$

3. Head loss due to bends (90^o; 45^o bend)

From Curve "C"

-- using largest velocity between V_7 and V_6 for 45^o bend

45^o $H_L = \underline{0.10 \text{ ft.}}$

-- using largest velocity between V_8 and V_6 for 90^o bend

90^o $H_L = 0.10 \times 2.0 = \underline{0.20 \text{ ft.}}$

4. Head loss due to secondary flows

From Curve "D"

$$\left.\begin{array}{l} Q_7 = 1.86 \\ Q_8 = 1.68 \end{array}\right] \text{ secondary flows}$$

$Q_y = 13.02$ upstream flow

$$\frac{Q_7}{Q_y} = \frac{1.86}{13.04} = 0.14 = 14\%$$

-- using V_4 for largest velocity

$H_L = \underline{0.08 \text{ ft.}}$

$$\frac{Q_8}{Q_y} = \frac{1.68}{13.02} = 0.13 = 13\%$$

H_L = $\underline{0.07 \text{ ft.}}$

Total H_L = $\underline{0.74 \text{ ft.}}$

Friction Head Losses:

Friction head losses are computed by the formula:

$$H_f = S_f \times L$$

where Hf = head loss in feet

L = pipe length in feet

$$S_f = \frac{Q \times N^2}{1.486\ (AR^{2/3})}$$

The friction head losses for each pipe section in the sample problem are shown below.

1 - 2 $S_f = \dfrac{7.53(0.013)^2}{1.486(0.566)} = 0.014$

$H_f = 0.014 \times 20.5' = 0.28$ ft.

2 - 3 $S_f = \dfrac{8.93(0.013)^2}{1.486(0.919)} = 0.007$

$H_f = 0.007 \times 178.1' = 1.25$ ft.

3 - 4 $S_f = \dfrac{10.64(0.013)^2}{1.486(0.919)} = 0.01$ ft.

$H_f = 0.01 \times 17' = 0.17$ ft.

5 - 4 $S_f = \dfrac{2.40(0.013)^2}{1.486(0.566)} = 0.001$

$H_f = 0.001 \times 17.5' = 0.02$ ft.

$$4 - 6 \quad S_f = \frac{13.04(0.013)^2}{1.486(1.979)} = 0.003$$

$$H_f = 0.003 \times 92' = 0.31 \text{ ft.}$$

$$7 - 6 \quad S_f = \frac{1.86(0.013)^2}{1.486(0.566)} = 0.001$$

$$H_f = 0.001 \times 167' = 0.167 \text{ ft.}$$

$$8 - 6 \quad S_f = \frac{1.68(0.013)2}{1.486(0.566)} = 0.001$$

$$H_f = 0.001 \times 11' = 0.01 \text{ ft.}$$

$$6 - 9 \quad S_f = \frac{16.58(0.013)^2}{1.486(3.588)} = 0.002$$

$$H_f = 0.002 \times 31' = 0.06 \text{ ft.}$$

Hydraulic Gradient Summary

The analysis of the hydraulic gradient computations is best performed with a summary sheet as shown in Figure 6-4. This permits easy comparison of hydraulic elevations with flowline elevations and the elevations of top of grates. The analysis is helped visually by plotting profile views (such as Figures 6-5, 6-6 and 6-7) showing the relative elevations of the various elements.

Gutter Flow Depths and Spreads

The curb, gutter and inlet hydraulics are analyzed to determine their capacities, flow depth and spread. Such evaluations are made below for inlets 2 and 7.

HYDRAULICS OF INLET 2 (45^o tilt-bar grate -- 1.7' x 3')

$Q = 1.4$ cfs

$d = .145'$, $T = 6.96$ ft., $S_x = 0.0208$,

$Z = 48$, $S_o = 0.013$

From Figure 6-26a:

$$\frac{W}{L'} = 0.31 \qquad W = 0.93$$

W_E = effective width = W + W = 1.7 + 0.93 = 2.63'

$$E_o = \text{Inlet efficiency} = 1 - \quad 1 - \frac{W_E}{T} \quad ^{8/3} = 1 - \quad 1 - \frac{2.63}{6.96} \quad ^{8/3} = 0.72$$

From Figure 6-28:

$K = 1.23$

$$V_F = \text{Frontal Velocity} = \frac{2KQZ}{T^2} = \frac{(2)(1.23)(1.4)(48)}{(6.96)^2} = 3.47 \text{ f/s}$$

From Figure 6-26b:

R = Reduction factor = 1.0

E = Hydraulic efficiency = E_o R = (0.72)(1) = 0.72

Q_i = Inlet capacity = EQ = (0.72)(1.4) = 1.01 cfs

Q_c = Carryover = Q - Q_i = (1.4 - 1.01) = 0.39 cfs

CK safety factor against clogging

$$\bar{d} = (T - \frac{W}{2})\ S_x = (6.96 - \frac{1.7}{2})(0.0208) = 0.127$$

$\bar{d}$ = (depth at mid point of grate)

$$V_F = \frac{QE_o}{w\bar{d}} = \frac{(1.4)(0.72)}{(1.7)(0.127)} = 4.67 \text{ f/s}$$

From Figure 6-26b:

1.7 feet of the grate is used at $V_F = 6.3$ f/s

$$S.F. = \frac{(6.3)}{(4.67)} = 1.3 \text{ against clogging}$$

HYDRAULICS OF INLET 7: (45° tilt-bar grate -- 1.7' x 3')

$Q = 1.79$ cfs, $d = 0.165'$, $T = 7.92'$
$S_x = 0.0208$ f/f, $Z = 48$

From Figure 6-26a:

$$\frac{W}{L_1} = 0.31, \quad W = 0.93$$

$$W_E = 2.63, \quad E_o = 1 - \left(1 - \frac{2.63}{7.92}\right)^{8/3} = 0.66$$

From Figure 6-28:

$K = 1.24$

$$V_F = \frac{(2)(1.24)(1.79)(48)}{(7.92)^2} = 3.40 \text{ f/s}$$

From Figure 6-26b:

$R = 1.0$

$E = (0.66)(1) = 0.66$
$Q_i = EQ = (0.66)(1.79) = 1.18$
$Q_c = (1.79 - 1.18) = 0.61$ cfs
CK against clogging

$$\bar{d} = (7.92 - \frac{1.7}{2})(0.0208) = 0.147$$

$$V_F = \frac{(1.79)(0.06)}{(1.7)(0.147)} = 4.73 \text{ f/s}$$

$$S.F. = \frac{6.1}{4.73} = 1.33 \text{ against clogging}$$

Probable Depth at Sumps

Inlet 1 is in a pocket at the lower end of a swale. Inlets 3 and 8 are at the pavement sag. Inlet 5 is at the sag but in the intercepting ditch on the south side of the roadway. The probable depths of design flow at each sump are computed as follows:

INLET I (Std. PW-BD-1 grate -- 2.37' x 3.5')

Given: Q = 7.53 cfs, S = 0.013

Perimeter = 2B + 2L = (2)(2.37) + (3.5)(2) = 11.74'
1/2 perimeter = 5.87

$$Q_p = \frac{7.53}{5.87} = 1.28 \text{ cfs/f}$$

From Figure 6-16:

d = 0.5'

(This is satisfactory since grate is pocketed at end of swale.)

INLET 3 (Std. grate -- 1.7' x 3')

Given: S = 0.013

$$\begin{aligned} Q_i &= (0.449)(3.85) && = 1.73 \text{ cfs} \\ Q_c &\text{ (carryover from Inlet 2)} && = \underline{0.39} \\ &\quad Q \text{ Total} && = 2.12 \text{ cfs} \end{aligned}$$

Perimeter = (2)(1.7) + 3.5 = 6.9 f
1/2 P for clogging = 3.45 f

$$Q_p = \frac{2.12}{3.45} = 0.61 \text{ cfs/f}$$

d = 0.37' = Depth of water above grate

INLET 5 (Std. PW-BD grate -- 2.37' x 3.5')

Given: $Q = 2.4$ cfs, $S = 0.013$

Perimeter = 11.74 x 1/2 P = 5.87

$$Q_P = \frac{2.4}{5.87} = 0.40$$

d = 0.30 (O.K. since at end of swale)

INLET 8 (Std. grate -- 1.7' x 1.3')

Given: $Q_8 = 1.68$, $S = 0.013$

Perimeter = 6.9 f.

1/2 P = 3.45 f.

$Q_c = 0.61$, $Q_T = 2.29$, $Q_P = .667$

d = 0.38 (O.K. since at end of swale)

Subsurface Drainage

The purpose of subsurface drainage is to remove detrimental groundwater in order to assure a stable roadbed and side slopes. Most often, those points or areas that require specific subsurface drainage will be determined during the process of field investigations. However, there are times when water-bearing formations are not revealed until construction has begun. "Bleeding" backslopes and apparent spring interceptions are two of the more obvious indications of this excessive groundwater.

Information on the potential need for underdrain treatment comes from several sources. The Materials Laboratory provides information on existing water tables and soils conditions. Maintenance personnel will recognize and identify unstable roadbed and surfacing conditions attributable to groundwater. And designers may locate potential trouble spots during field reviews.

Perforated pipe underdrain should be installed to intercept most subsurface drainage waters. These pipes, available in sizes of 6-inch, 8-inch and larger diameters, may form a network of interceptors and be routed to a central point of collection and outfall. Installation of underdrains should be as shown in the Standard Sheets.

Subsurface drainage is of primary importance in curb and gutter sections of the roadway. Where there is evidence of subsurface water, perforated underdrain pipe should be installed so as to empty into the drainage structure connecting the storm drain pipe -- at an elevation well above the flow line and as high as practicable above the storm drain pipe. This will eliminate the possibility of perforated pipe emptying against a head developed in the drainage structure and forcing the water out the perforations.

Surface drainage should not be permitted to discharge into an underdrain. Underdrains should be permitted to empty into a roadway drainage system only where the outfall is not against a head.

Normally, underdrains of 6-inch diameter should be allowed to run no more than 500 feet. For distances over 500 feet, the minimum diameter should be 8 inches. A minimum of 0.20 percent pipe gradient is recommended.

Cleanouts should be installed on long runs of underdrain pipe. These may consist of a vertical small-diameter steel water pipe through which a pressure hose can be connected to flush out the system. The vertical pipe should be capped. Outlets for underdrains normally should be spaced no greater than 1,000 feet.

Pipe Design

Previous sections discussed the determination of pipe sizes in relation to estimated design discharge. This section is directed to design considerations related to physical standards for pipes and to criteria for installation.

Types of Pipe

Pipes are manufactured in various sizes and shapes and from various types of materials. Types of pipe commonly used in Delaware include:

- Reinforced Concrete Pipe (Round)
- Reinforced Concrete Pipe (Horizontally Elongated)
- Corrugated Steel Pipe (Round)
- Corrugated Steel Pipe Arch
- Corrugated Aluminum Pipe (Round)
- Corrugated Aluminum Pipe Arch
- Corrugated Steel Pipe Downspouts
- Perforated Pipe Underdrains

Large-size corrugated metal structural plate pipes also are available, but usually they are a special design responsibility of the Bridge Section.

Pipe arches and horizontally elongated pipes normally are used only where there is a limited amount of headroom and minimum cover over the culvert.

For corrugated steel pipes, several optional special treatments may be called for:

- Bituminous Coated -- a complete coating of bituminous material over galvanized steel.

- Bituminous Coated with Paved Invert -- additional protection for the pipe invert, coated over galvanized steel.

- Full Paved -- paving the inside total circumference of galvanized steel pipe to the depth of corrugations.

- Aluminized -- a thin coating of aluminum over steel (instead of galvanizing) as protection against corrosion.

The bituminous and aluminum coatings provide greater resistance to corrosion than is found with plain galvanized pipe. They are desirable with conditions of corrosive soils. For extremely corrosive conditions, corrugated aluminum pipe should be considered. The soils report provides a measure of corrosiveness of the soils along with guidelines for the most appropriate treatment.

The paved invert provides added protection to the flowline from the erosive action of sand and debris during high velocity flow. Additionally, a fully paved pipe demonstrates better hydraulic characteristics with a lower value of Manning's roughness coefficient.

The flowline slope of the pipe may influence the type of pipe. Concrete pipes normally should not be used when the velocity is over 10 feet/second. Straight-line metal pipes may be used with higher velocities, but consideration should be given to the use of stepped pipes and downspouts.

The type of pipe to be used should be based on the desirable hydraulic characteristics, soil conditions under which it must function, and the installation and maintenance cost. In Delaware, where the slopes of the pipes in a storm drainage system are relatively flat and pipe depths are minimal, concrete pipe will usually best meet the above requirements, and is desirable when placed under the pavement of roadway and shoulders.

In the design of drainage systems in Delaware, it is recommended that concrete pipe be given prime consideration for all pipes, size 15" to 48" diameter, that are to be placed under roadway pavements. Pipes larger than 48" should be given special consideration as to type, end protection, velocity, bedding, soil condition, etc. Pipes placed under entrances, private drives, access roads with low traffic volumes or roads with dirt or surface-treated surfaces may be either concrete or corrugated metal, provided hydraulic and soil conditions can be met.

Strength Requirements

Pipes are manufactured to several different strength standards.

Reinforced concrete pipe comes in three classes. Class III is considered the standard strength and is used unless a stronger pipe is called for. Classes IV and V have thicker walls and more reinforcing steel and should be used where warranted for conditions of unusual loading or high fills.

The strength of corrugated steel and corrugated aluminum pipe is measured in terms of the gage (thickness of metal). Corrugated metal pipe also comes with three different sizes of corrugation -- 2-2/3" x 1/2," 3" x 1," and 5" x 1" -- where the first number is the distance between crests and the second is the depth of corrugations. The larger corrugation produces greater strength than the smaller -- with the same gage metal. At present, the 3" x 1" and 5" x 1" corrugations are used only for pipes of 36-inch diameter and larger.

The factor that primarily influences the strength requirements for pipes is the height of fill above the top of the pipe. Figures 6-29 and 6-30 identify the strength requirements for corrugated steel pipes and corrugated steel pipe arches. Maximum permissible cover is shown for various combinations of pipe diameter, pipe shape and thickness of metal. Maximum cover is based on the distance from the top of the pipe to the elevation of the finished road surface.

The figures also show minimum permissible cover for the various types and shapes of pipes. Minimum cover is measured from the top of the pipe to the top of the subgrade, since this will be the effective cover during construction operations.

Figures 6-31 and 6-32 show comparable strength requirements for corrugated aluminum pipe and corrugated aluminum pipe arches.

Strength requirements for concrete pipe culverts are shown in Figure 6-33 in terms of maximum permissible cover for each of the three classes of pipe. The minimum cover over concrete pipe is one foot.

Class III is the standard design for reinforced concrete pipes. When there is need to specify a stronger pipe, the pipe class must be clearly identified on the plans.

All depth-of-cover limitations are based on use of Type C bedding for culverts.

Culvert End Section Treatment

Usually some type of special treatment is warranted at the ends of culverts. Department Standard Sheets show details for end section treatments for both concrete and metal pipes.

Flared end sections or beveled ends should be considered for most pipe installations with a skew of 30 degrees or less. They provide for better entrance and exit flow characteristics -- and, since they fit closely to the roadway slope, they are less of an obstruction to a vehicle leaving the roadway surface.

Safety end structures (see Standard Sheets) should be provided at the ends of culverts through median crossovers. These end structures include metal grates on a relatively flat slope and serve as a safety feature for vehicles accidentally leaving the roadway and entering the median.

Special treatment often is needed at culvert ends as erosion control measures for protection of the embankment. Criteria for plain riprap and sack riprap are shown in the Standard Sheets.

Concrete headwalls at culvert ends are recognized as a potential traffic safety hazard, particularly when they are located close to the roadway shoulder. Generally, concrete headwalls should be avoided unless they are required for:

1. unique hydraulic conditions, or
2. structural support for very large metal culverts.

In the case of relatively high fills and steep side slopes, the terminal section of pipe may be left with a square end and no special end section treatment. The square end should extend about one foot beyond the intersection of the side slope with the natural ground. Normally, some type of erosion control protection should be considered for these installations.

Multiple Pipe Installations

Where two or more pipes are placed side by side, the spacing between adjacent pipes should be sufficient to permit effective compaction with hand tamps. Normally, a minimum separation of 3 feet is desirable. Closer spacing is permitted for unusual circumstances.

Skewed Installations

Wherever practicable, pipe culvert installations should be designed to conform as closely as possible to the natural drainage channels.

The degree of skew is measured as the angle between the pipe installation and a line perpendicular to the highway centerline. A culvert angle is described in terms of which end is forward -- left forward or right forward. For example, if the left end of the culvert is ahead of the line perpendicular to the centerline, and the angle is 15 degrees, the installation would be described as "15^{0} skew left forward."

Culvert Length Measurement

For estimating purposes, culvert lengths are measured along the culvert flow line. To avoid the need for cutting sections, the design length should be in increments of two feet wherever practicable. When an installation requires that a section be cut (such as a storm sewer installation), payment will be for the actual length required.

Flared end sections are not considered a part of the culvert length. They are measured and paid for by the number of end sections installed.

Permanent Erosion Control

Most highway projects will require certain erosion control measures as a protection for roadside ditches and slopes. This section discusses those control measures of a permanent nature that are incorporated in project design. A following section will discuss temporary erosion control measures that must be considered by designers.

Delaware law prescribes certain policies and procedures with regard to erosion and sedimentation control. As a guide for implementing these requirements, the Delaware Department of Natural Resources and Environmental Control has published the "Erosion and Sediment Control Handbook." Reference should be made to this publication for more details on both permanent and temporary erosion control actions.

Vegetation

Barren soils are especially susceptible to erosion by wind and water. Established vegetation is the most natural and effective means of erosion protection on roadside slopes. It is the basic policy of the Department that vegetation be restored to all areas disturbed during construction operations. Designers should consider the following requirements.

1. Topsoil. Prior to general grading operations, the contractor must remove existing topsoil from the areas to be disturbed. Such topsoil is stockpiled for future placement following completion of grading operations. If required, additional topsoil may be obtained from other sources. Quantity estimates must be prepared in terms of the square yards of area to be topsoiled.

2. Seeding. Seeding of all topsoil areas is prescribed in accordance with the criteria shown in the Standard Specifications. The quantity of seeding is estimated as the square yards of surface area actually covered.

3. Mulching. Mulch must be applied to all seeded areas to prevent water or wind erosion. Quantity estimates should be prepared in terms of square yards for:

   - mulching, straw or hay,
   - mulching, wood cellulose fibre,
   - mulching, woven paper, excelsior or jute.

   Additional quantity estimates must be made for securement of mulching, as follows.

   - mulch securement, chemical mulch binder (sq. yd.). (straw or hay mulch)
   - mulch securement, mulch crimper (sq. yd.). (straw or hay mulch)
   - asphalt mulch binder (gallons)

4. Sodding. Sodding should be called for at locations where it would be extremely difficult to effectively restore vegetation with normal seeding practices. Quantity estimates for sodding should be in terms of square yards.

Ditch Treatment

Ditch channels are particularly susceptible to erosion and often require some type of special erosion control measure. A principal factor in determining the type of treatment is the velocity of the water flow. General criteria for the various types of treatment are shown in Figure 6-34.

With relatively low velocities, well-established grass usually will provide adequate protection. This may be accomplished through normal seeding and/or sodding of the roadside. There may be need for some interim temporary erosion control measures (see following sections). With flat terrain and extremely low velocities (0.5 cfs or less), concrete lining may be considered to expedite the flow and dispersal of water.

With higher velocities it is necessary to consider a ditch lining of riprap or some other permanent protection. Details of these treatment are shown in the Department's Standard Sheets.

Fill Slope Protection

Where roadways and fill slopes are adjacent to a body of water and are subject to erosion, the slopes may be protected with stone riprap. This consists of toe trenches filled with a filter blanket of sand-gravel as a base for dumped stone riprap placed on the slope. (This procedure is not to be used where sinusoidal wave action erosion is encountered.)

On other high fills, consideration should be given to the construction of bituminous curbs and corrugated metal downspouts as shown in the Standard Sheets.

Other Erosion Control Devices

1. Interception Ditches -- Small ditches and berms immediately above the top of cut slopes for the purpose of intercepting surface runoff and carrying the water to a natural channel for disposal.

2. Culvert Riprap -- Stone or broken concrete riprap placed at the inlets and/or outlets of pipe culverts, possibly including a riprap apron at the outlet.

3. Energy Dissipators -- Several types of impact basins, drop structures or stilling wells at the outlets of culverts for the purpose of reducng or eliminating the effects of erosion. These are usually special-design items for unusually high velocities and local conditions particularly susceptible to erosion. Reference is made to Hydraulic Engineering Circular No. 14, "Hydraulic Design of Energy Dissipators for Culverts and Channels" (FHWA, December 1975).

Temporary Erosion and Sedimentation Control

Newly constructed side slopes and ditches are particularly susceptible to erosion. There is need to plan temporary control measures to be implemented during construction operations, to serve as protection until the permanent erosion controls are installed and the slopes are stabilized.

Also, the construction activities bring about conditions which may contribute to contamination of nearby natural bodies of water. Temporary measures often are needed during construction to prevent this from happening.

Where there are identifiable conditions that warrant these temporary controls, designers should make certain that the appropriate controls are clearly shown on the plan sheets, along with any additional right of way or easements that may be required. Details of the various erosion control items are shown in the Standard Sheets and are briefly described below.

Dikes

Several types of temporary dikes commonly are used to control storm runoff during construction. Usually they consist of a ridge of compacted soil, and their use is limited to drainage areas of less than 5 acres. Typical installations include:

1. Diversion Dikes -- constructed immediately above a cut or fill slope to intercept storm runoff from higher areas and divert it away from exposed slopes to a stabilized outlet.

2. Interceptor Dikes -- constructed across disturbed right of way and similar sloping areas to shorten the length of exposed slopes, thereby reducing the potential for erosion.

3. Perimeter Dikes -- constructed at the perimeter of the site to divert runoff from the construction area.

4. Straw (or Hay) Bale Dikes -- installed across, or at the toe of a slope to intercept and detain small amounts of sediment from unprotected areas of less than 1/2 acre. They should not be on high sediment producing areas, above "high risk" areas, where water is concentrated in a channel, or where there is possibility of a washout.

Swales

Swales serve a purpose similar to dikes -- but they consist of an excavated drainage way rather than a ridge of compacted soil. Details for interceptor swales and perimeter swales are shown on the Standard Sheets.

Stone Outlet Structure

Stone outlet structures may be installed in conjunction with dikes at locations where they will provide a protected outlet for controlled overflow. Crushed stone is used instead of the compacted soil used in the remainder of the dike.

Level Spreader

This provides an outlet for concentrated runoff by dispersing the water on a zero percent grade at non-erosive velocities onto undisturbed area that is stabilized by existing vegetation. The design length of the spreader should be based on the estimated Q_{10}, as indicated on the Standard Sheet.

Sediment Traps

A sediment trap is a temporary basin of limited capacity formed by excavation and/or embankment. It is used to intercept sediment-laden storm runoff and to trap and retain the sediment in order to protect drainage ways, properties and rights of way below the trap. The drainage area for a sediment trap should not exceed 5 acres.

Sediment traps should be designed with dimensions to assure a volume (measured at the elevation of the crest of the outlet) of at least 1800 cubic feet per acre of drainage area. Embankments for sediment traps shall not exceed 5 feet in height as measured at the low point of the original groundline -- and shall have a minimum top width of 4 feet.

Each sediment trap shall be clearly shown on the plans, along with the following information:

1. type of trap,
2. size of outlet,
3. trap dimensions,
4. embankment height and depth of excavation, and
5. drainage area.

Storm Inlet Sediment Trap

This trap provides temporary protection against sedimentation entering storm inlets during construction operations. It consists of a basin surrounding or adjacent to newly constructed storm inlets. Details are shown on the Standard Sheet.

Grade Stabilization Structures

These are temporary structures placed from the top to the bottom of a slope to convey surface runoff down the slope without causing erosion. The Standard Sheets show three optional designs, using rigid pipe slope drains, flexible pipe slope drains, or a paved chute or flume -- along with design criteria for sizes of pipe as related to drainage areas.

LIST OF FIGURES

Chapter Six

Figure 6-1

PLAN OF STORM DRAINAGE SYSTEM - EXAMPLE PROBLEM

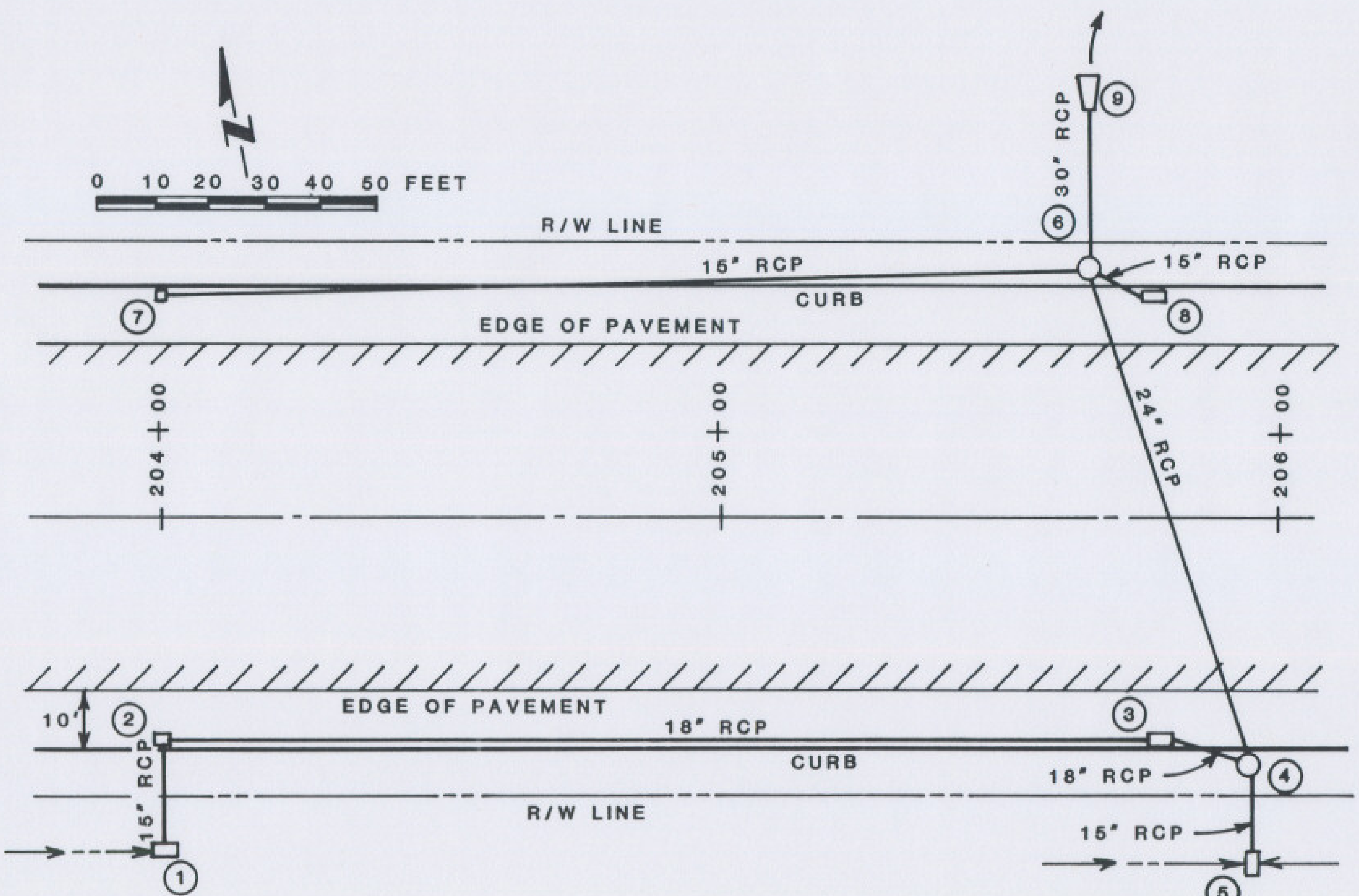
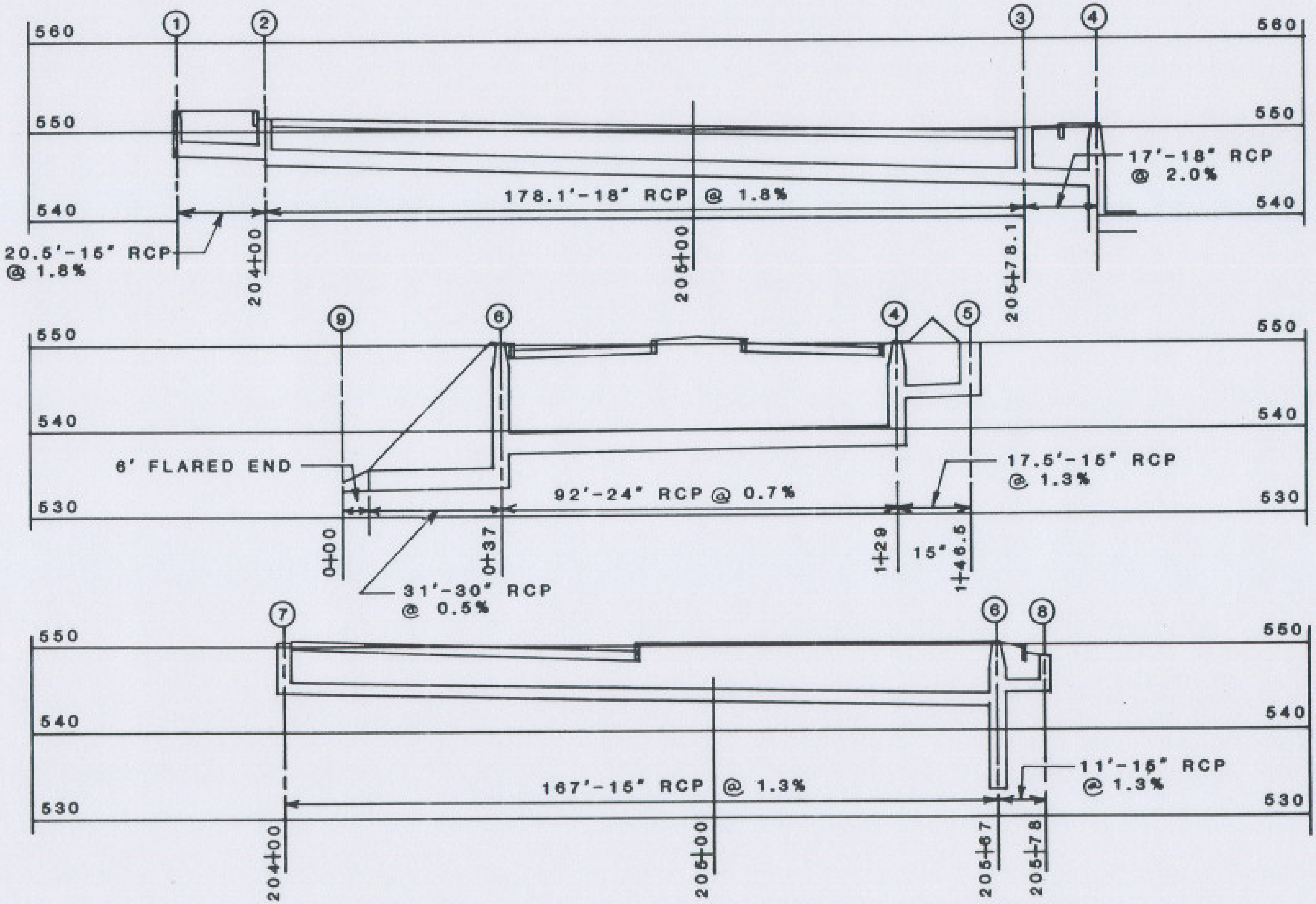


Figure 6-2

PROFILE OF STORM DRAINS - EXAMPLE PROBLEM



NOTE: ELEVATIONS ARE SHOWN ON FIGURES 6-5 THROUGH 6-7.

Figure 6-3

STORM DRAINAGE DESIGN

By: Nat Date: ______

Checked by: D. Ho Date: ______ Location: Design Manual Sheet No. ______ Of ______

Storm Frequency: ______ ______ Contract No. ______

Location		Increment Area	Time: Sum of Time to Inlet	Time: Flow Time in Pipe	Rainfall Intensity	Runoff Coefficient		ΣΔC.A.	Discharge	Length of Pipe	Slope	Roughness Coefficient	Pipe Size	Mean Velocity	Just Full Capacity	Invert Upper End	Invert Lower End	~~Remarks~~ Actual Velocities
		ΔA	T	T	I	C	ΔC.A.	C.A.	Q	L	S	N		V	Q			
From	To	Acres	Min.	Min.	In./Hr.		Acres	Acres	C.F.S.	FT.	FT./FT.		IN.	F.P.S.	C.F.S.	FT.	FT.	
1	2	2.59	26.5	0.04	3.85	0.755	1.96	1.96	7.53	20.5	0.018	0.013	15	7.07	8.68	47.34	46.97	7.99
2	3	0.48	26.54	0.04	3.85	0.757	0.363	2.32	8.93	178.1	0.018	0.013	18	6.78	11.98	46.72	44.40	7.53
3	4	0.54	26.44	0.03	3.80	0.831	0.449	2.77	10.64	17.0	0.020	0.013	18	8.41	14.86	44.20	43.86	9.25
5	4	0.95	26.0	0.07	3.92	0.645	0.613	0.613	2.40	17.5	0.020	0.013	15	4.40	9.27	44.00	43.65	5.08
4	6	—	26.97	0.21	—	—	—	3.38	13.04	92.0	0.007	0.013	24	6.02	18.92	38.00	37.36	6.50
7	6	0.50	26.0	0.81	3.92	0.950	0.475	0.475	1.86	167.0	0.009	0.013	15	4.82	5.90	44.50	43.00	4.14
8	6	0.46	26.5	0.04	3.85	0.950	0.437	0.437	1.68	11.0	0.020	0.013	15	7.19	8.82	44.74	44.63	5.20
6	9	—	27.18	0.12	—	—	—	—	16.58	31.0	0.005	0.013	30	5.9	29.0	33.16	33.00	6.10

Figure 6-4

HYDRAULIC GRADIENT COMPUTATIONS

Location		Pipe Size	Length of Pipe	Pipe Slope	Hydraulic Slope S_f	H.L. In. Str.	Friction H.L.	Flow Line Elevation (Ft.)		Hydraulic Elevation (Ft.)		Top of Grade
From	To	In.	Ft.	Ft./Ft.	Ft./Ft.	Ft.	Ft.	Upper	Lower	Upper	Lower	Ft.
1	2	15	20.5	0.018	0.014	1.34	0.28	47.34	46.97	50.28	48.66	52.30
2	3	18	178.1	0.018	0.007	0.81	1.25	46.72	44.40	48.66	46.60	51.44
3	4	18	17.0	0.020	0.001	1.07	0.17	44.20	43.86	46.60	45.36	49.66
5	4	15	17.5	0.020	0.001	0.55	0.20	44.00	43.65	45.65	44.90	50.00
4	6	24	92.0	0.007	0.003	0.57	0.31	38.00	37.36	40.24	39.36	50.10
7	6	15	167.0	0.009	0.001	0.38	0.17	44.50	43.00	44.80	44.25	50.52
8	6	15	11.0	0.020	0.001	0.59	0.01	44.74	44.63	46.48	45.88	48.74
6	9	30	31.0	0.005	0.002	0.74	0.06	33.16	33.00	36.30	35.50	50.16

After hydraulic elevation is determined at structures check vs. top of grate elevation to ensure against blow out of drainage structures. It is desirable that the hydraulic gradient not be within 1.5 ft. of the surface.

Figure 6-5

HYDRAULIC GRADIENT PROFILES
EXAMPLE PROBLEM

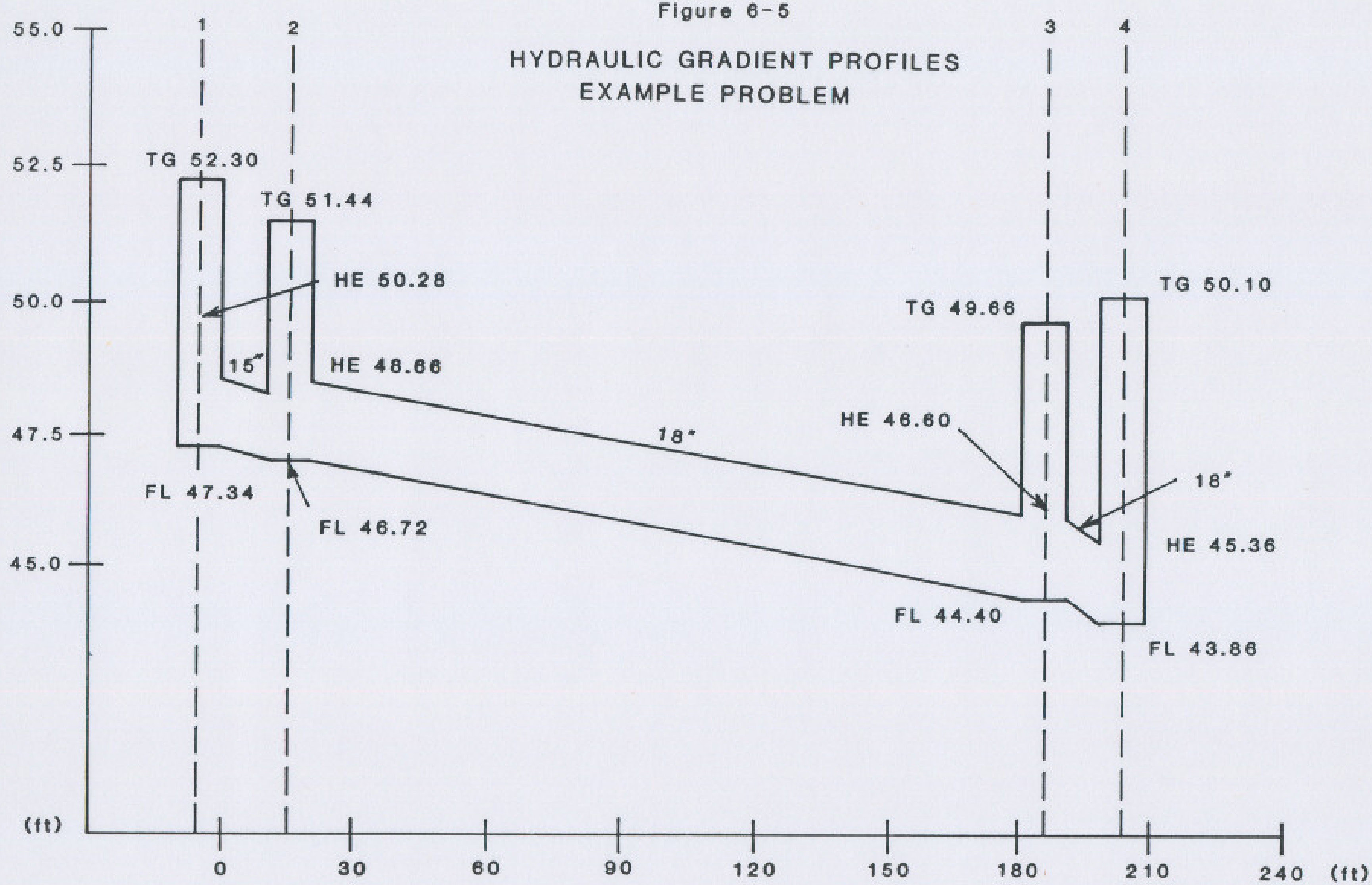


Figure 6-6

HYDRAULIC GRADIENT PROFILES — EXAMPLE PROBLEM

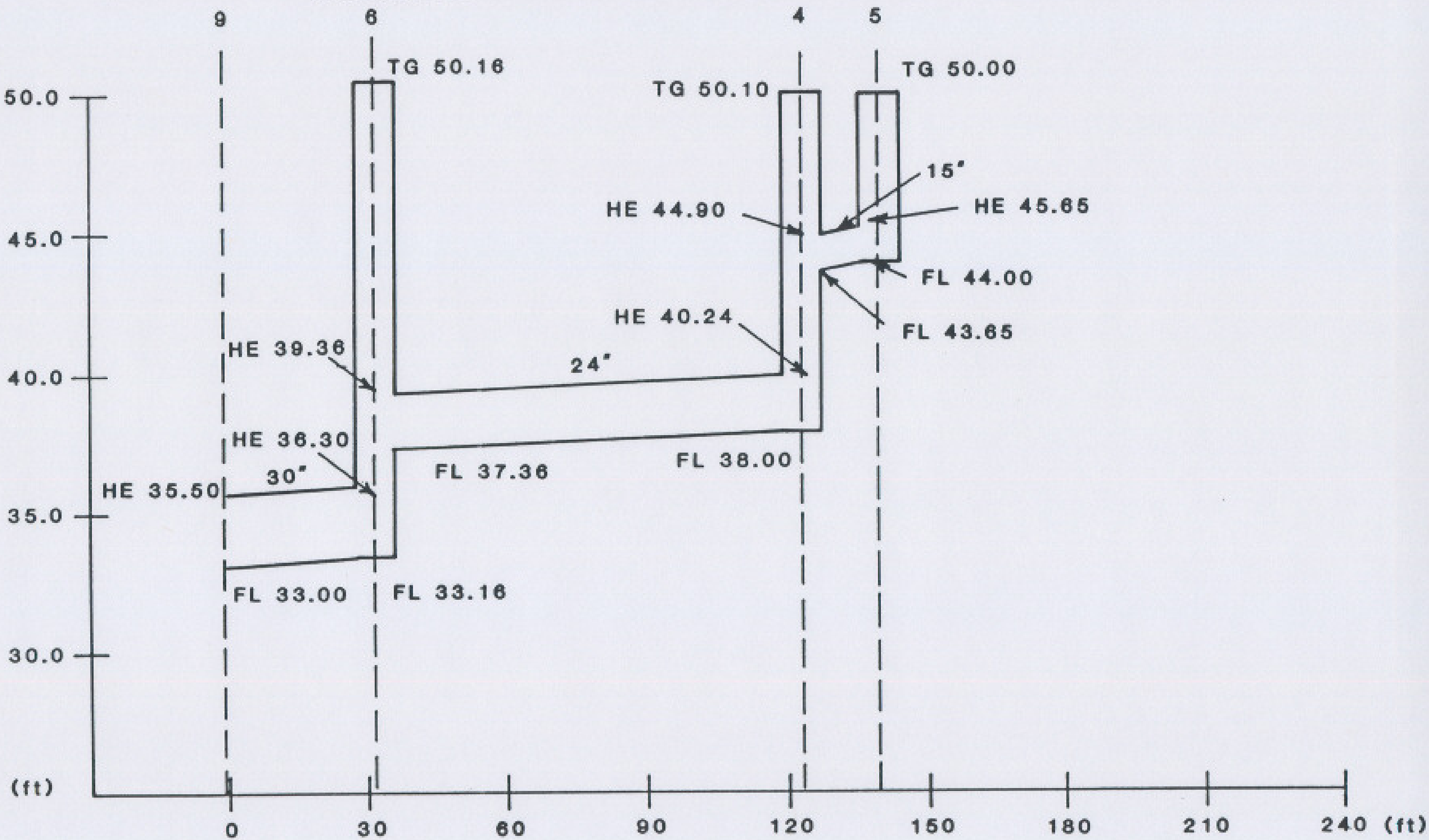


Figure 6-7
HYDRAULIC GRADIENT PROFILES
EXAMPLE PROBLEM

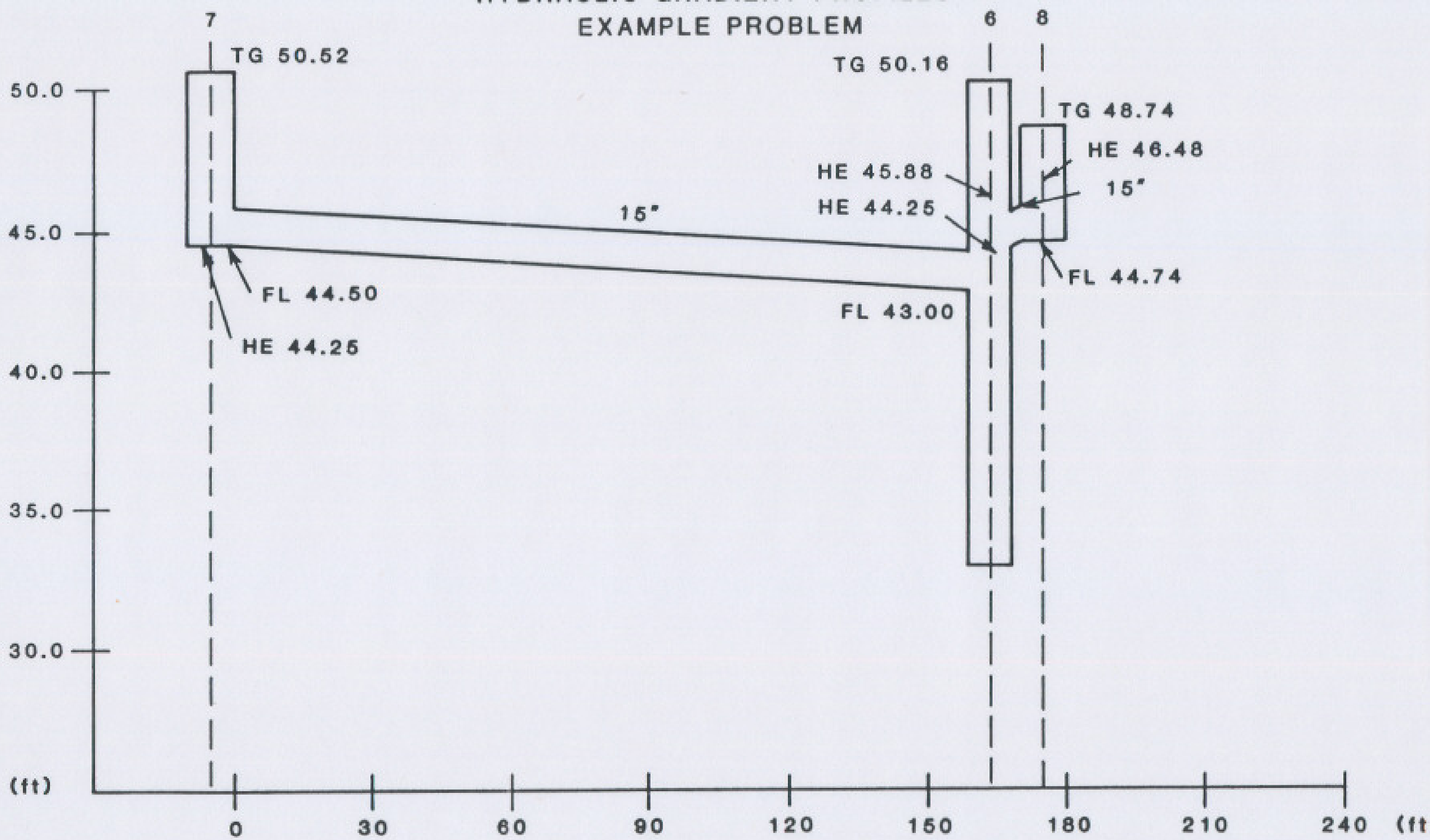


Figure 6-8

CRITERIA FOR STORM DESIGN FREQUENCY

Road Classification			STORM DESIGN YEARS				
			Bridges (Over 20' Span)	Culverts	Storm Drains	Side Ditches	Medians
Rural	Arterials		50	25	10	10	10
Rural	Collectors	Over 2,000 ADT	50	25	10	10	10
Rural	Collectors	Over 2,000 ADT	25	10	5	5	5
Rural	Local		25	10	5	5	5
Urban	Arterials		50	50	10	10	10
Urban	Collectors		50	25	10	10	10
Urban	Local		50	25	10	5	10

Figure 6-9

Compiled by ____________

Date ____________________

DESIGN RISK ASSESSMENT

(Based upon design engineering data)

1. Identify involvement within the base flood plain ______________________

____

____

2. Traffic Service: ADT ___________

   Detours Available _______________ Length _____________

   Overtopping flood Q = ___________ E.P. ____________%

   Stage ___________

   Potential damage to the highway facility _____________

   ____

3. Applicable Flood Plain Management Criteria ____________________________

____

____

4. Note social, economic, ecological and human use of flood plain land ____

____

____

____

5. Drainage Area ____

6. Compare the hydraulic performance of the proposed action to the hydraulic performance of the existing conditions in terms of:

   Changes in flood level

   Changes in flood velocities

   Changes in flood flow distribution

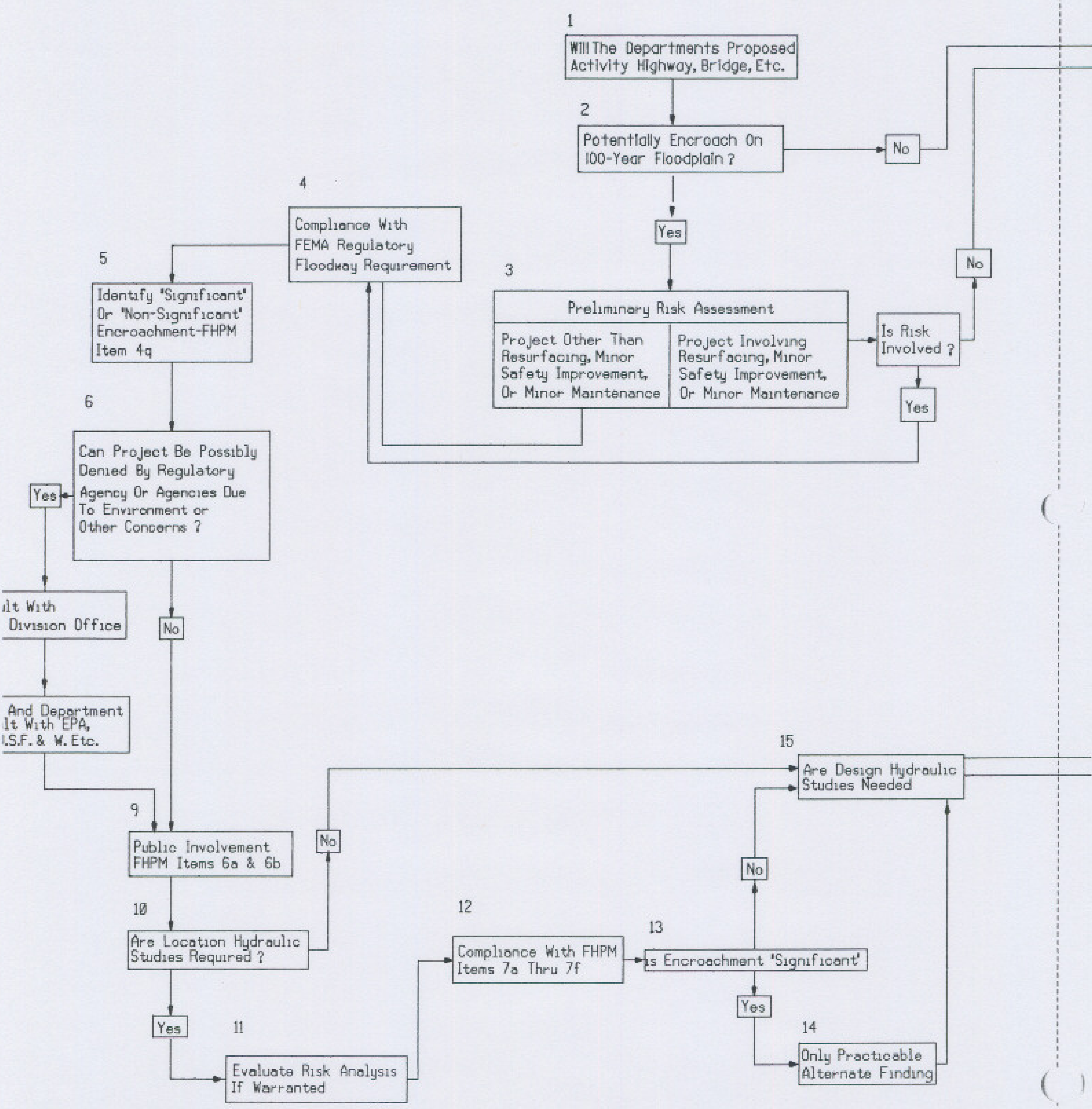
   Changes in impoundment (retention) characteristics

   Changes in hydraulic structure configuration

7. Remarks

If significant changes in hydraulic performance and/or "Risk" are present, an economic risk analysis (LTEC) may be required.

PROCEDURES FOR COMPLIANCE WITH WATERWAY AND FLOODPLAIN MANAGEMENT REGULATIONS



PROCEDURES FOR COMPLIANCE WITH WATERWAY AND FLOODPLAIN MANAGEMENT REGULATIONS

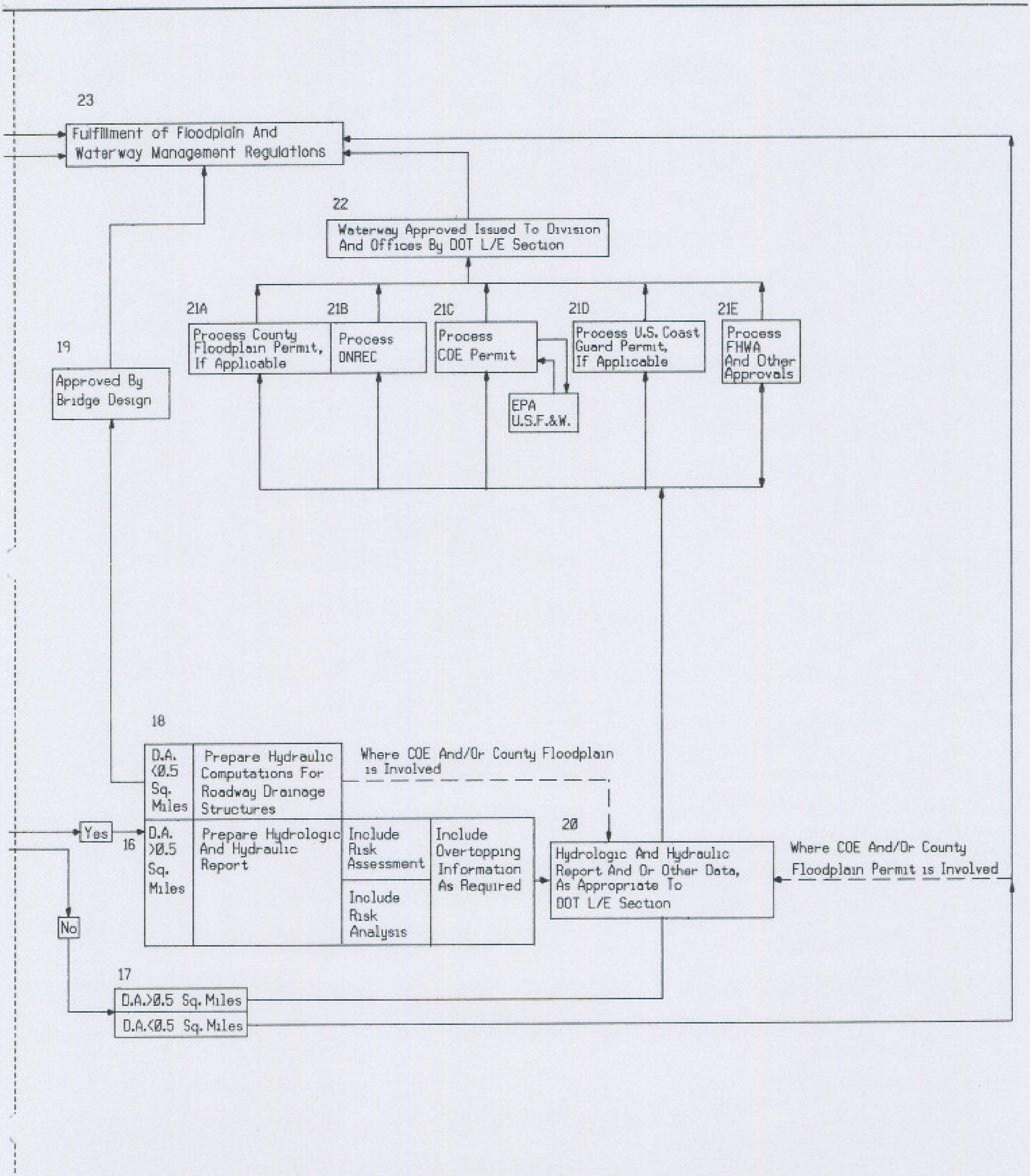


Figure 6-11

VALUES OF RUNOFF COEFFICIENT "C"

TOPOGRAPHY 7 VEGETATION	SOIL TEXTURE OPEN SANDY LOAM	CLAY 7 SILT LOAM	TIGHT CLAY
WOODLAND			
Flat 0-5% Slope	0.10	0.30	0.40
Rolling 5-10% Slope	0.25	0.35	0.50
Hilly 10-30% Slope	0.30	0.50	0.60
PASTURE			
Flat 0-5% Slope	0.10	0.30	0.40
Rolling 5-10% Slope	0.16	0.36	0.55
Hilly 10-30% Slope	0.22	0.42	0.60
CULTIVATED			
Flat 0-5% Slope	0.30	0.50	0.60
Rolling 5-10% Slope	0.40	0.60	0.70
Hilly 10-30% Slope	0.52	0.72	0.82

URBAN AREAS	30% IMPERVIOUS	50% IMPERVIOUS	70% IMPERVIOUS
Flat 0-5% Slope	0.40	0.55	0.75
Rolling 5-10% Slope	0.50	0.65	0.80

PAVED AREAS	
Bituminous & Concrete Pavement	c = 0.90 - 1.00
Densely Built Paved Areas	c = 0.90
Commercial & Factory Areas, Road Shoulders	c = 0.70
Light Factories & Apartment Areas	c = 0.60
Compact Residential Areas	c = 0.50
Suburban Residential Areas	c = 0.30 - 0.40
Gravel Surfaces	c = 0.25 - 0.70
Parks, Railways Yards	c = 0.20

NOTE: Values of "C" can be combined to get a weighted average for a watershed consisting of several distinct zones.

e.g. Watershed consists of:

20% Hilly Woodland on sandy loam (c = 0.30)
30% Rolling Woodland on clay and sill loam (c = 0.35)
50% Rolling Cultivated tight clay (c = 0.70)

Average "C" (0.2 x 0.30) + (0.3 + 0.35) + (0.5 x 0.70) = 0.52

Figure 6-12

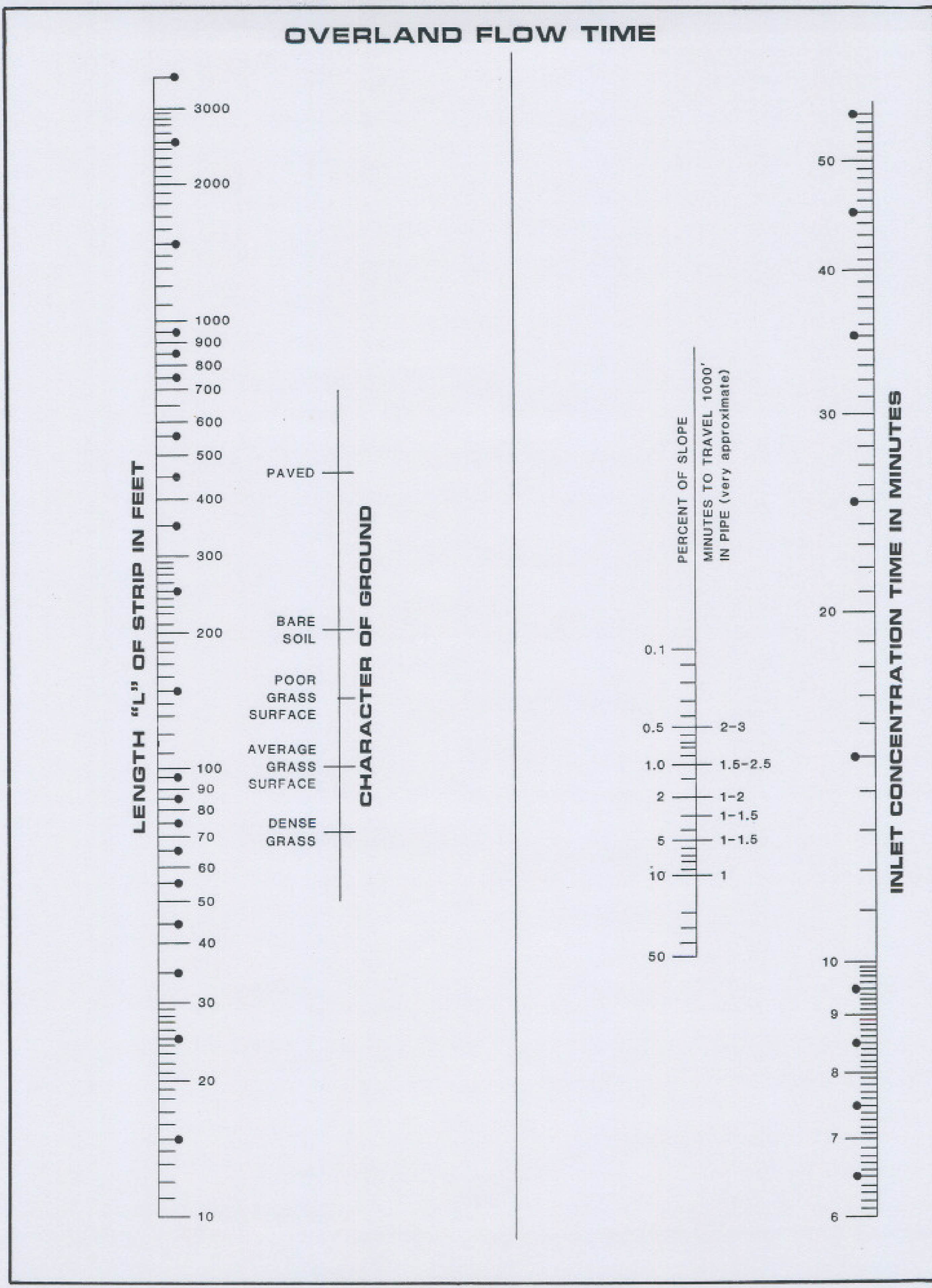

OVERLAND FLOW TIME
LENGTH "L" OF STRIP IN FEET
3000
2000
1000
900
800
700
600
500
400
300
200
100
90
80
70
60
50
40
30
20
10
CHARACTER OF GROUND
PAVED
BARE SOIL
POOR GRASS SURFACE
AVERAGE GRASS SURFACE
DENSE GRASS
PERCENT OF SLOPE
MINUTES TO TRAVEL 1000' IN PIPE (very approximate)
0.1
0.5
1.0
2
5
10
50
2-3
1.5-2.5
1-2
1-1.5
1-1.5
1
INLET CONCENTRATION TIME IN MINUTES
50
40
30
20
10
9
8
7
6

Figure 6-13

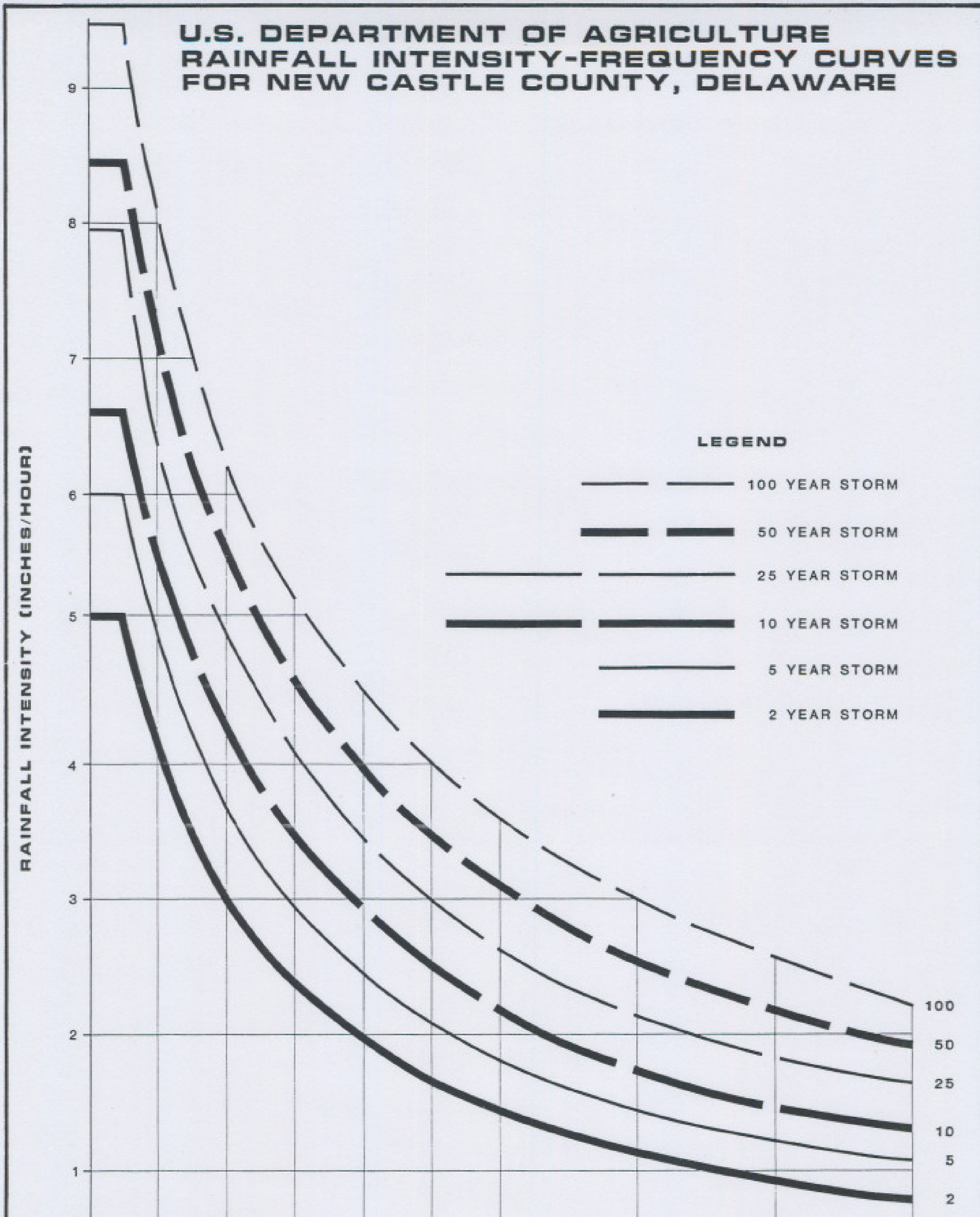
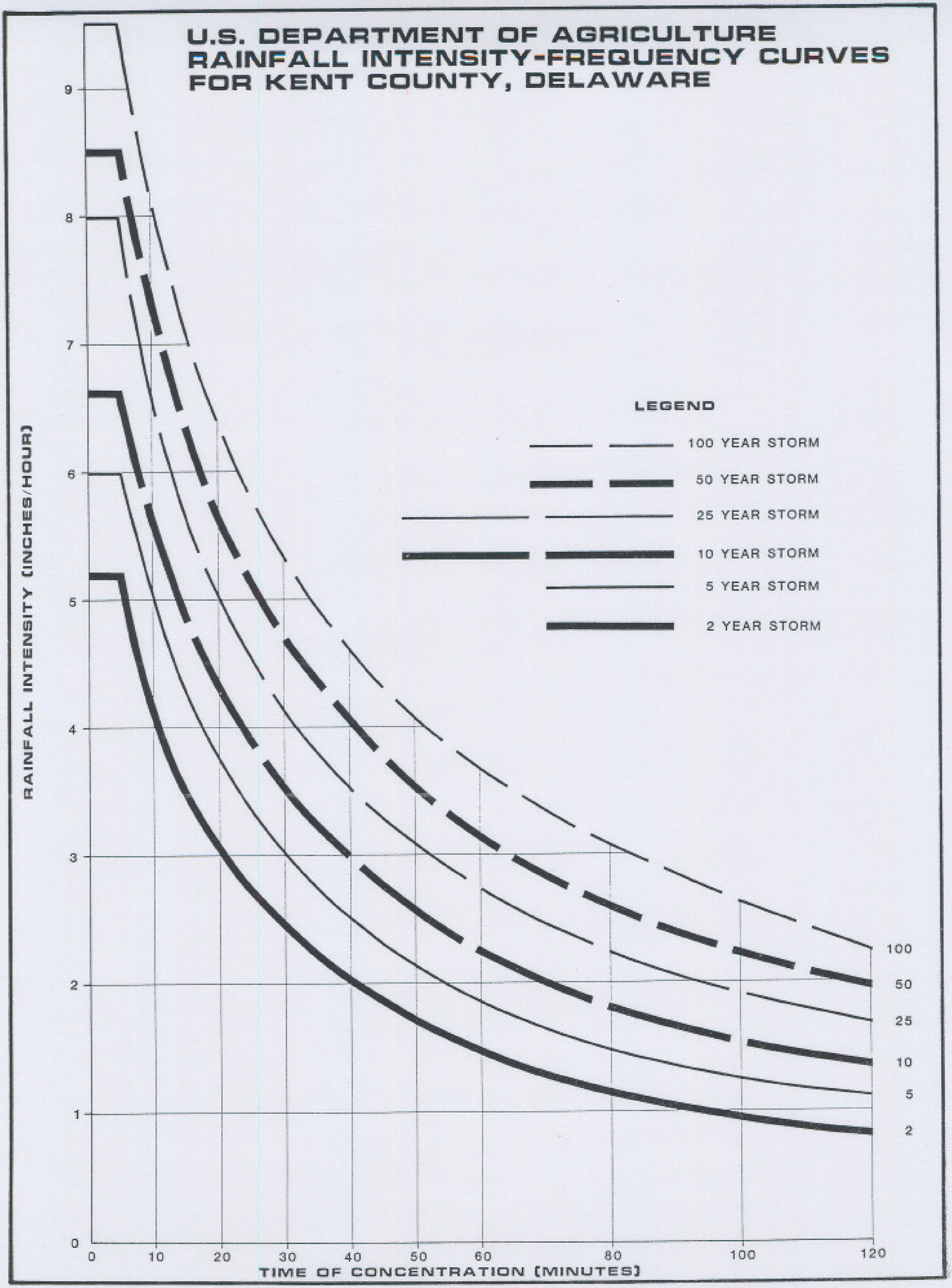
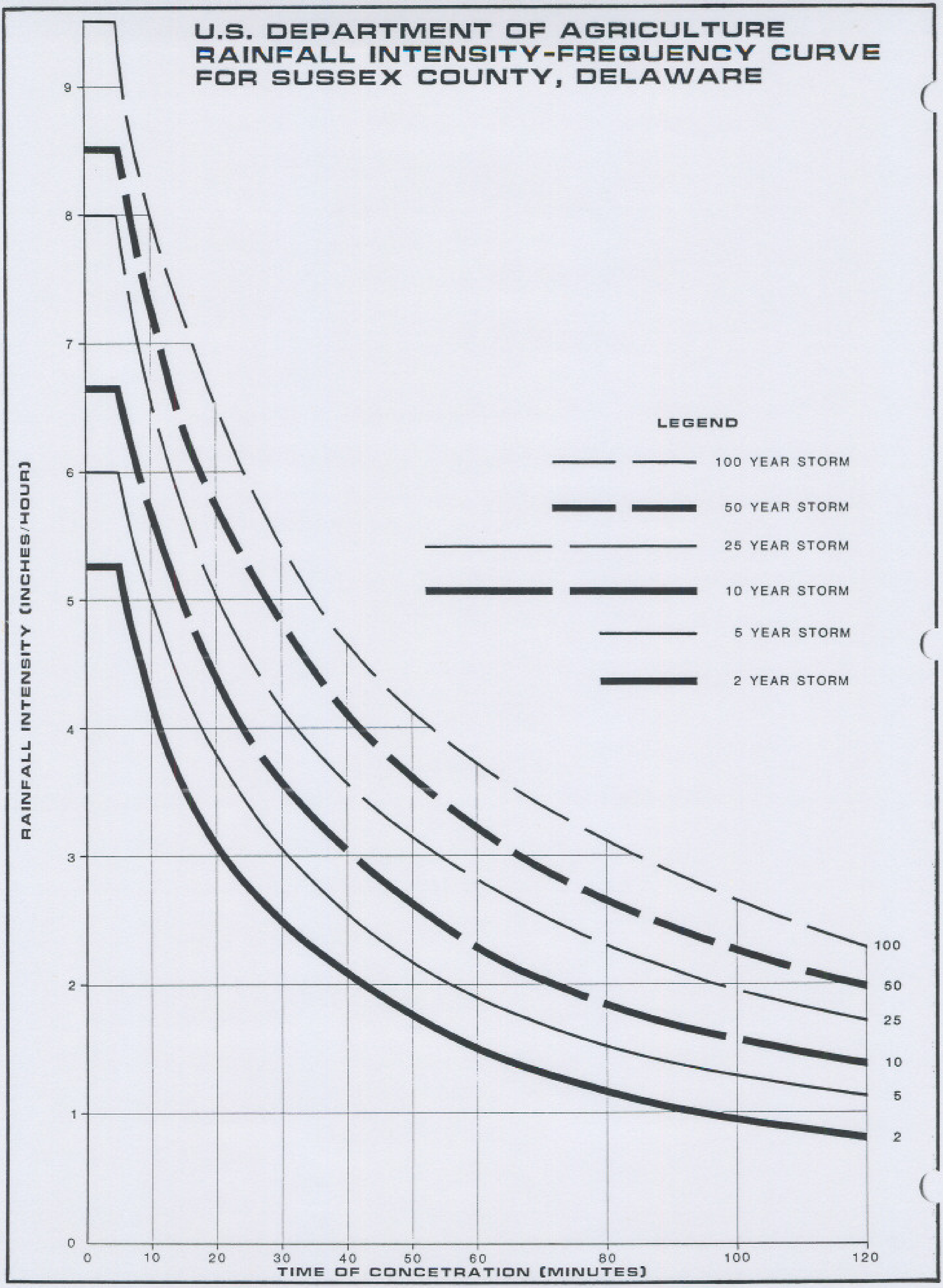

U.S. DEPARTMENT OF AGRICULTURE
RAINFALL INTENSITY-FREQUENCY CURVES
FOR NEW CASTLE COUNTY, DELAWARE
LEGEND
100 YEAR STORM
50 YEAR STORM
25 YEAR STORM
10 YEAR STORM
5 YEAR STORM
2 YEAR STORM
RAINFALL INTENSITY (INCHES/HOUR)
TIME OF CONCENTRATION (MINUTES)
0
1
2
3
4
5
6
7
8
9
10
20
30
40
50
60
80
100
120
100
50
25
10
5
2

Figure 6-14

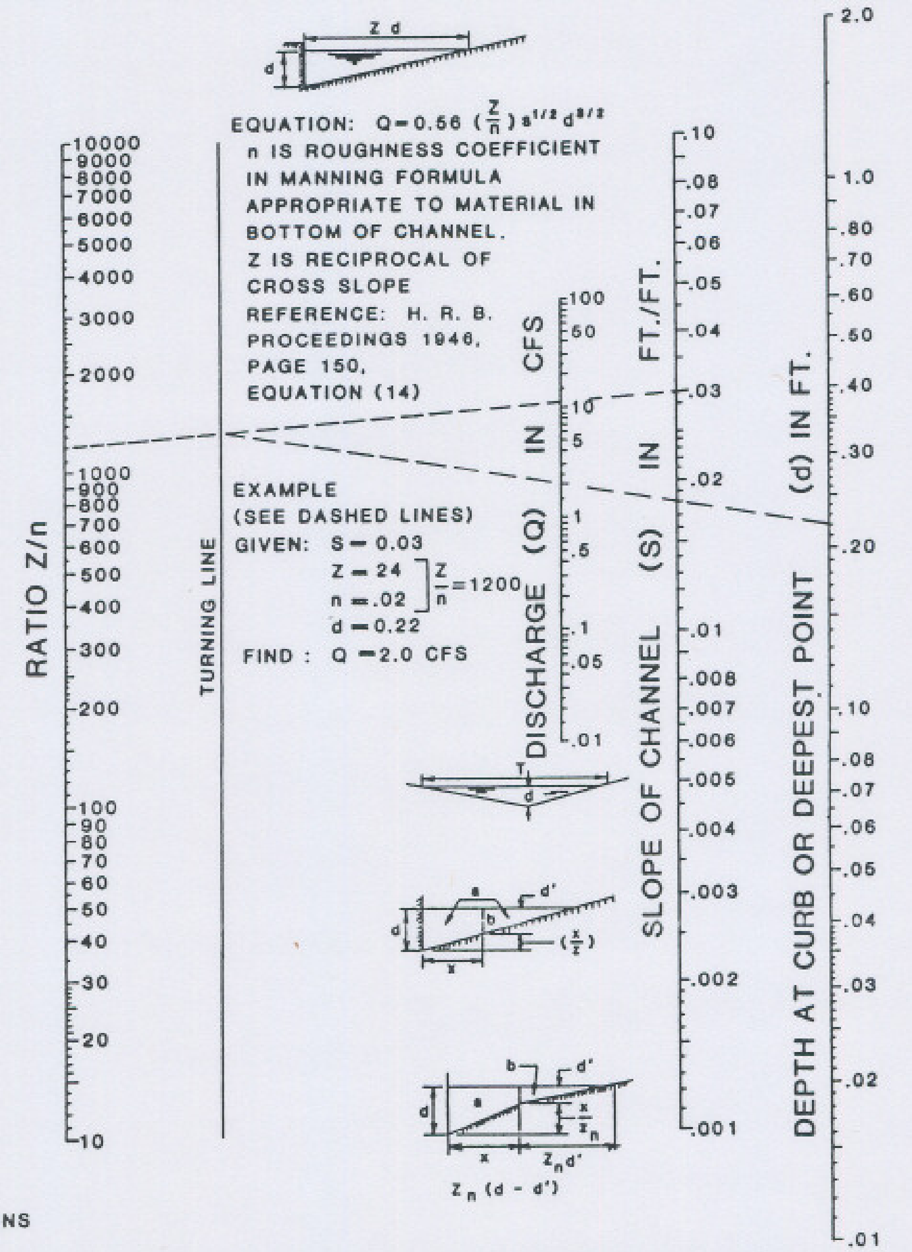


U.S. DEPARTMENT OF AGRICULTURE
RAINFALL INTENSITY-FREQUENCY CURVES
FOR KENT COUNTY, DELAWARE
LEGEND
100 YEAR STORM
50 YEAR STORM
25 YEAR STORM
10 YEAR STORM
5 YEAR STORM
2 YEAR STORM
RAINFALL INTENSITY (INCHES/HOUR)
0
1
2
3
4
5
6
7
8
9
TIME OF CONCENTRATION (MINUTES)
0
10
20
30
40
50
60
80
100
120
100
50
25
10
5
2

Figure 6-15


U.S. DEPARTMENT OF AGRICULTURE
RAINFALL INTENSITY-FREQUENCY CURVE
FOR SUSSEX COUNTY, DELAWARE
LEGEND
100 YEAR STORM
50 YEAR STORM
25 YEAR STORM
10 YEAR STORM
5 YEAR STORM
2 YEAR STORM
RAINFALL INTENSITY (INCHES/HOUR)
0
1
2
3
4
5
6
7
8
9
0
10
20
30
40
50
60
80
100
120
TIME OF CONCETRATION (MINUTES)
100
50
25
10
5
2

NOMOGRAPH FOR GUTTER CAPACITY



INSTRUCTIONS

1. CONNECT Z/n RATIO WITH SLOPE (S) AND CONNECT DISCHARGE (Q) WITH DEPTH (d). THESE TWO LINES MUST INTERSECT AT TURNING LINE FOR COMPLETE SOLUTION.
2. FOR SHALLOW V-SHAPED CHANNEL AS SHOWN USE NOMOGRAPH WITH $Z=\frac{T}{d}$
3. TO DETERMINE DISCHARGE Q_z IN PORTION OF CHANNEL HAVING WIDTH X: DETERMINE DEPTH d FOR TOTAL DISCHARGE IN ENTIRE SECTION a. THEN USE NOMOGRAPH TO DETERMINE Q_h IN SECTION b FOR DEPTH $d'=d-(\frac{X}{Z})$
4. TO DETERMINE DISCHARGE IN COMPOSITE SECTION : FOLLOW INSTRUCTION 3. TO OBTAIN DISCHARGE IN SECTION a AT ASSUMED DEPTH d: OBTAIN Q_b FOR SLOPE RATIO Z_b AND DEPTH d'. THEN $Q_t=Q_a+Q_b$

Figure 6-17

COEFFICIENT OF ROUGHNESS VALUES
(Manning's "n")

CORRUGATED METAL PIPE:

Corrugations	Annular 2 2/3 x 1/2 In.	Helical 1 1/2 x 1/4 in (11.12)		Helical 2 2/3 x 1/2 in (13)					
	All Diameters	8"	10"	12"	18"	24"	36"	48"	60" and Larger
Unpaved	.024	.012	.014	.011	.014	.016	.019	.020	.021
25% Paved	.021					.015	.017	.020	.019
Fully Paved	.012					.012	.012	.012	.012

	Annular 3 x 1 In.	Helical - 3 x 1 in.					
		48"	54"	60"	66"	72"	78" and Larger
Unpaved	.027	.023	.023	.024	.025	.026	.027
25% Paved	.023	.020	.020	.021	.022	.022	.023
Fully Paved	.012	.012	.012	.012	.012	.012	.012

Corrugations 6 x 2 in.	60"		72"		120"		180"
Plain -- Unpaved	.033		.032		.030		.028
25% Paved	.028		.027		.026		.024

* AISI

CONCRETE PIPE: "n" = 0.013

Figure 6-18

CONVEYANCE FACTORS -- ROUND PIPE

Dia., in.	A Area, sq. ft.	Various Values of n																	
		.010	.012	.013	.014	.016	.017	.018	.019	.020	.021	.023	.024	.025	.031	.032	.033	.034	.035
6	.20	7.29	6.08	5.61	5.21	4.56													
8	.35	15.70	13.10	12.10	11.20	9.82													
10	.55	28.50	23.70	21.90	20.30	17.80													
12	.79	46.30	38.60	35.60	33.10	28.90													
15	1.23	84.00	70.00	64.60	60.00	52.50													
18	1.77	137	114	105	98	85	80	76	72	68	65	59	57	55					
21	2.41	206	172	158	147	129	121	114	108	103	98	90	86	82					
24	3.14	294	245	226		184	173	163	155	147	140	128	123	118					
30	4.91	533	444	410		333	314	296	281	267	254	232	222	213					
36	7.07	867	723	667			510	482	456	434	413	377	361	347					
42	9.62	1310	1090	1010				727	688	654	623	569	545	523					
48	12.57	1870	1560	1440						934	889	812	778	747					
54	15.90	2560	2130	1970						1280	1220	1110	1070	1020					
60	19.63	3390	2820	2600						1690	1610	1470	1410		1090	1060	1030	996	967
66	23.76	4370	3640	3360							2080	1900	1820		1410	1360	1320	1280	1250
72	28.27	5510	4590	4240							2620	2390	2290		1780	1720	1670	1620	1570
78	33.18	6820	5680	5240							3250	2960	2840		2200	2130	2070	2000	1950
84	38.48	8300	6920	6390							3950	3610	3460		2680	2600	2520	2440	2370
90	44.18	9980	8320	7680							4750	4340	4160		3220	3120	3020	2940	2850
96	50.27	11900	9880	9120							5650	5160	4940		3820	3710	3590	3490	3390
102	56.75	13900	11600	10700							6640	6060	5810		4500	4360	4220	4100	3980
108	63.62	16200	13500	12500							7730	7060	6760		5240	5070	4920	4770	4640
114	70.88	18700	15600	14400							8930	8150	7810		6050	5860	5680	5510	5360
120	78.54	21500	17900	16500							10200	9350	8960		6930	6720	6510	6320	6140
126	86.59	24500	20400	18800											7900	7650	7420	7200	7000
132	95.03	27700	23100	21300											8940	8660	8400	8150	7920
138	103.87	31200	26000	24000											10100	9750	9460	9180	8920
144	113.10	35000	29100	26900											11300	10900	10600	10300	9990
150	122.72	39000	32500	30000											12600	12200	11800	11500	11100
156	132.73	43300	36100	33300											14000	13500	13100	12700	12400
162	143.14	47900	39900	36800											15400	15000	14500	14100	13700
168	153.94	52700	43900	40600											17000	16500	16000	15500	15100
174	165.13	57900	48300	44500											18700	18100	17500	17000	16500
180	176.71	63400	52800	48800											20400	19800	19200	18600	18100

Figure 6-19

CONVEYANCE FACTORS -- CMP ARCH PIPE

Size, In.		A Area, sq. ft.	R Hydraulic Radius, ft.	Values of n									
Span	Rise			.012	.014	.016	.017	.018	.019	.020	.021	.024	.025
17	13	1.2	.311	69	59	52	49	46	44	42	40	35	33
21	15	1.7	.357	105	90	78	74	70	66	63	60	52	50
24	18	2.3	.415	157	135	118	111	105	99	94	90	79	75
28	20	2.9	.458	212	182	159	150	141	134	127	121	106	102
35	24	4.4	.563	373	320	280	263	249	236	224	213	187	179
42	29	6.5	.689	627	537	470	442	418	396	376	358	313	301
49	33	8.4	.766	873	748	655	616	582	551	524	499	436	419
57	38	11.3	.896	1300	1110	972	915	864	819	778	741	648	622
64	43	14.4	1.021	1810	1550	1360	1280	1210	1150	1090	1040	907	871
71	47	17.4	1.109	2310	1980	1730	1630	1540	1460	1390	1320	1160	1110
77	52	21.3	1.232	3030	2600	2270	2140	2020	1910	1820	1730	1520	1450
83	57	25.6	1.359	3890	3340	2920	2750	2600	2460	2340	2220	1950	1870

Figure 6-20

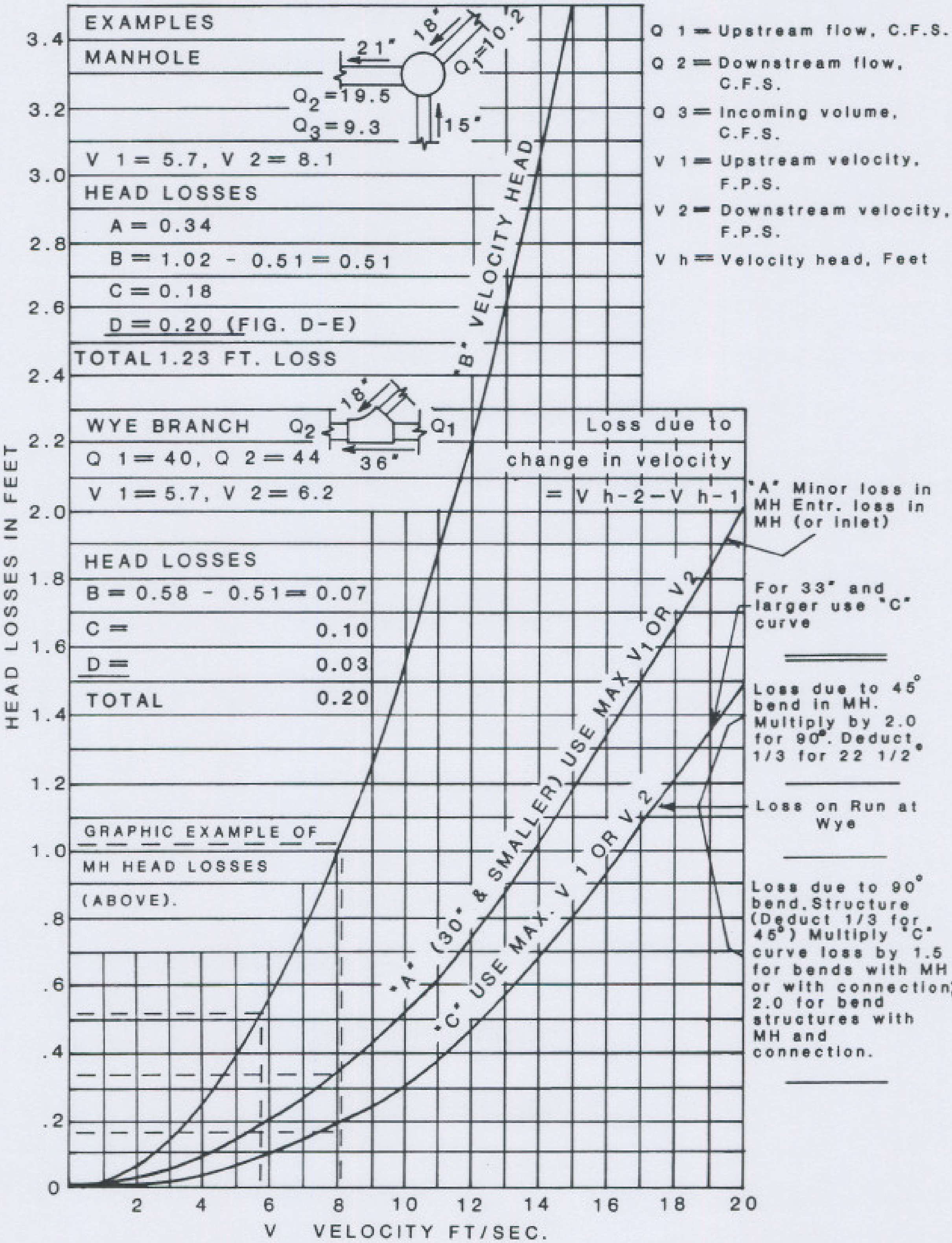
VALUES OF $S^{1/2}$ FOR USE IN MANNING'S EQUATION

S	0	1	2	3	4	5	6	7	8	9
.00	-	.0316	.0447	.0548	.0632	.0707	.0775	.0837	.0894	.0949
.01	.10	.1049	.1095	.1140	.1183	.1225	.1265	.1304	.1342	.1378
.02	.1414	.1449	.1483	.1517	.1549	.1581	.1612	.1643	.1673	.1703
.03	.1732	.1761	.1789	.1817	.1844	.1871	1.897	.1924	.1949	.1975
.04	.20	.2025	.2049	.2074	.2098	.2121	.2145	.2168	.2191	.2214
.05	.2236	.2258	.2280	.2302	.2324	.2345	.2366	.2387	.2408	.2429
.06	.2449	.2470	.2490	.2510	.2530	.2550	.2569	.2588	.2608	.2627
.07	.2646	.2665	.2683	.2702	.2720	.2739	.2757	.2775	.2793	.2811
.08	.2828	.2846	.2864	.2881	.2898	.2915	.2933	.2950	.2966	.2983
.09	.30	.3017	.3033	.3050	.3066	.3082	.3098	.3114	.3130	.3146
.10	.3162	.3178	.3194	.3209	.3225	.3240	.3256	.3271	.3286	.3302

Note: Numbers across top represent third decimal place, e.g. if given slopes are .002 ft./ft. and .034 ft./ft. then respective $S^{1/2}$ values are .0447 and .1844.

Figure 6-21

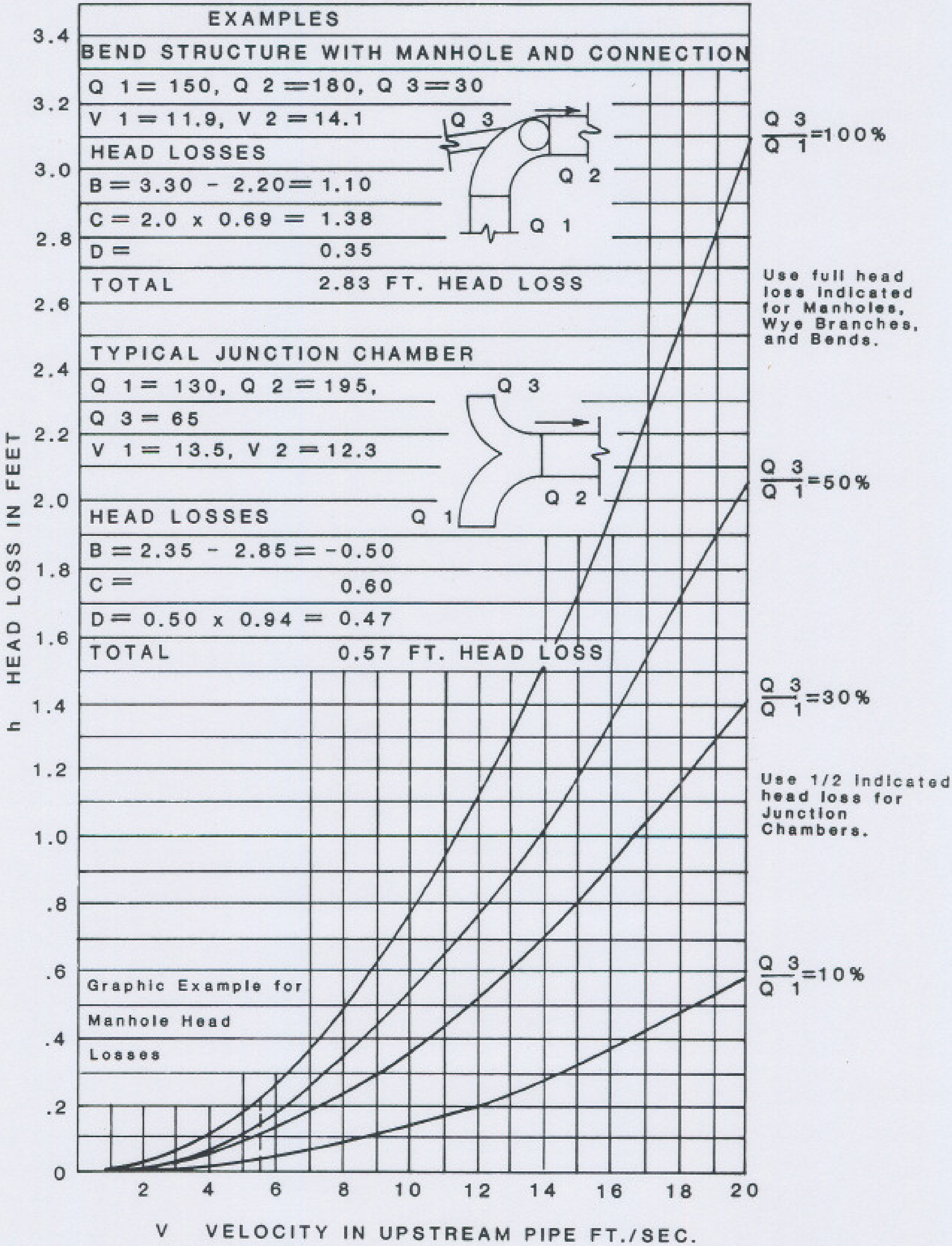
HEAD LOSSES IN STRUCTURES-"A", "B", & "C" LOSSES



Source: Baltimore County Department of Public Works

Figure 6-22

HEAD LOSSES IN STRUCTURE "D" LOSS



Source: Baltimore County Department of Public Works

Figure 6-23

HYDRAULIC CAPACITY OF GRATE INLET IN SUMP

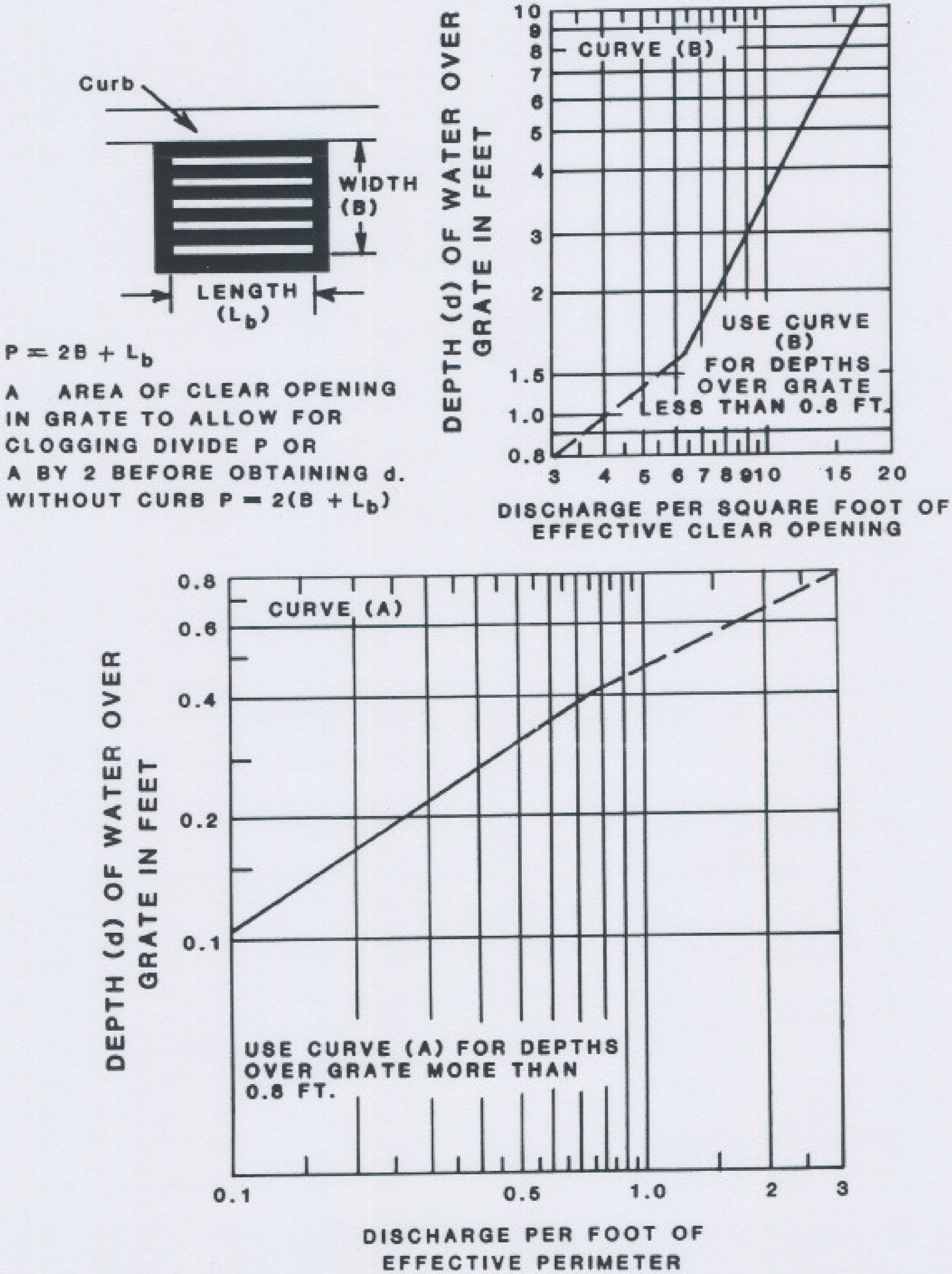
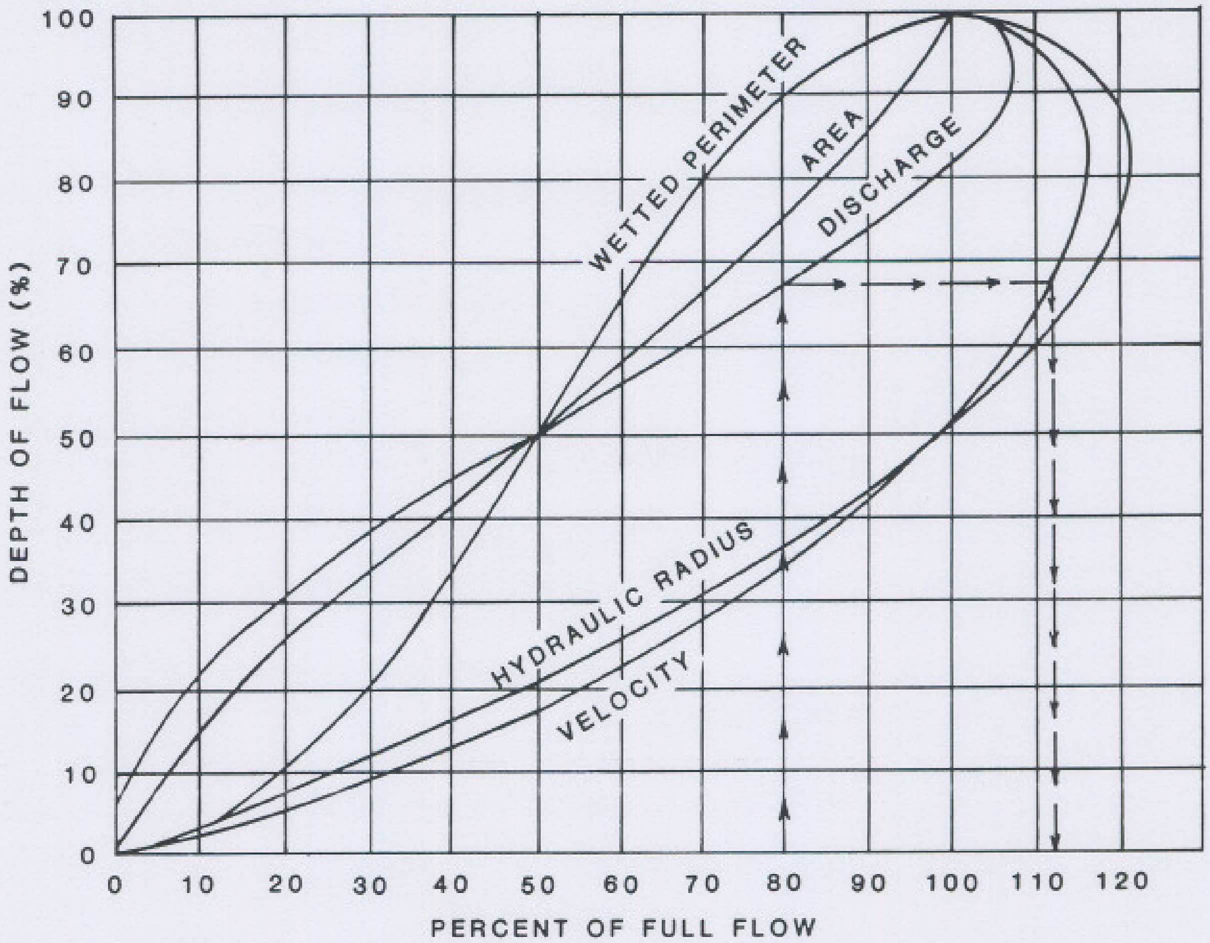


Figure 6-24

HYDRAULIC ELEMENTS OF CIRCULAR SECTIONS



EXAMPLE:

Given- Discharge flowing full 15 c.f.s., velocity = 7 f.p.s.

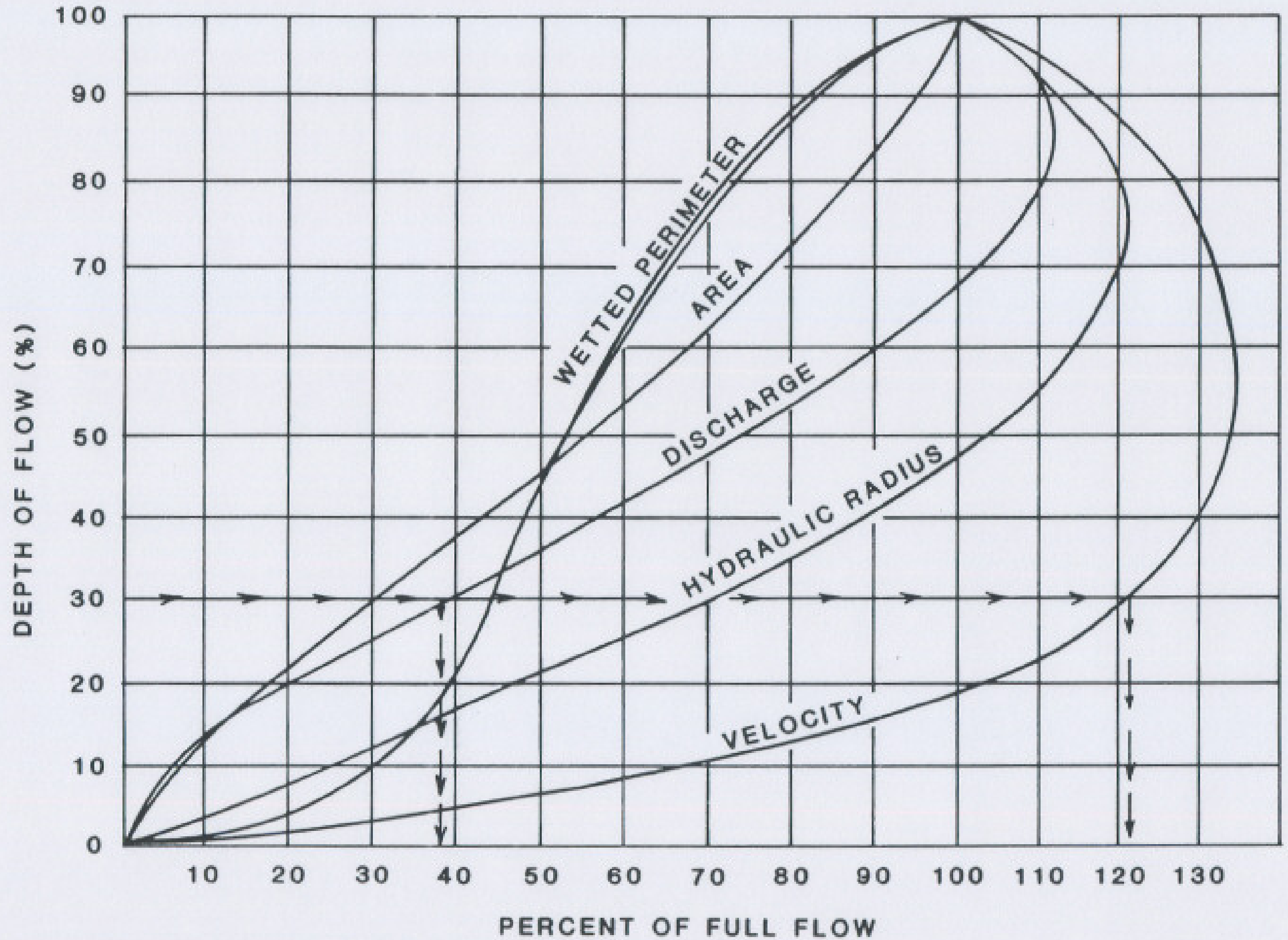
Determine- Velocity and depth of flow when discharge is 12 c.f.s.

Solution- Enter chart at 80% of value for full section of Hydraulic Elements. Obtain depth of flow 68% of full flow depth and velocity = 112.5% x 7 = 7.9 f.p.s.

Source: Baltimore County Department of Public Works

Figure 6-25

HYDRAULIC ELEMENTS OF ARCH PIPE SECTIONS



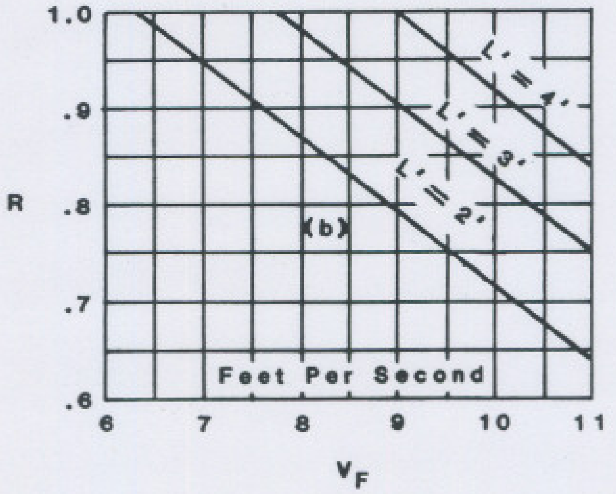
EXAMPLE: Given- Discharge flowing full 20 cfs ; velocity = 10 fps
Determine- Discharge & velocity when depth of flow is 30% of depth flowing full
Solution- Q = 38% of 20 cfs = 7.6 cfs V = 122% of 10 fps = 12.2 fps

Source: Baltimore County Department of Public Works

Figure 6-26

GRATE INLET DESIGN CURVES - STANDARD GRATES

45° Tilt Bar Grate



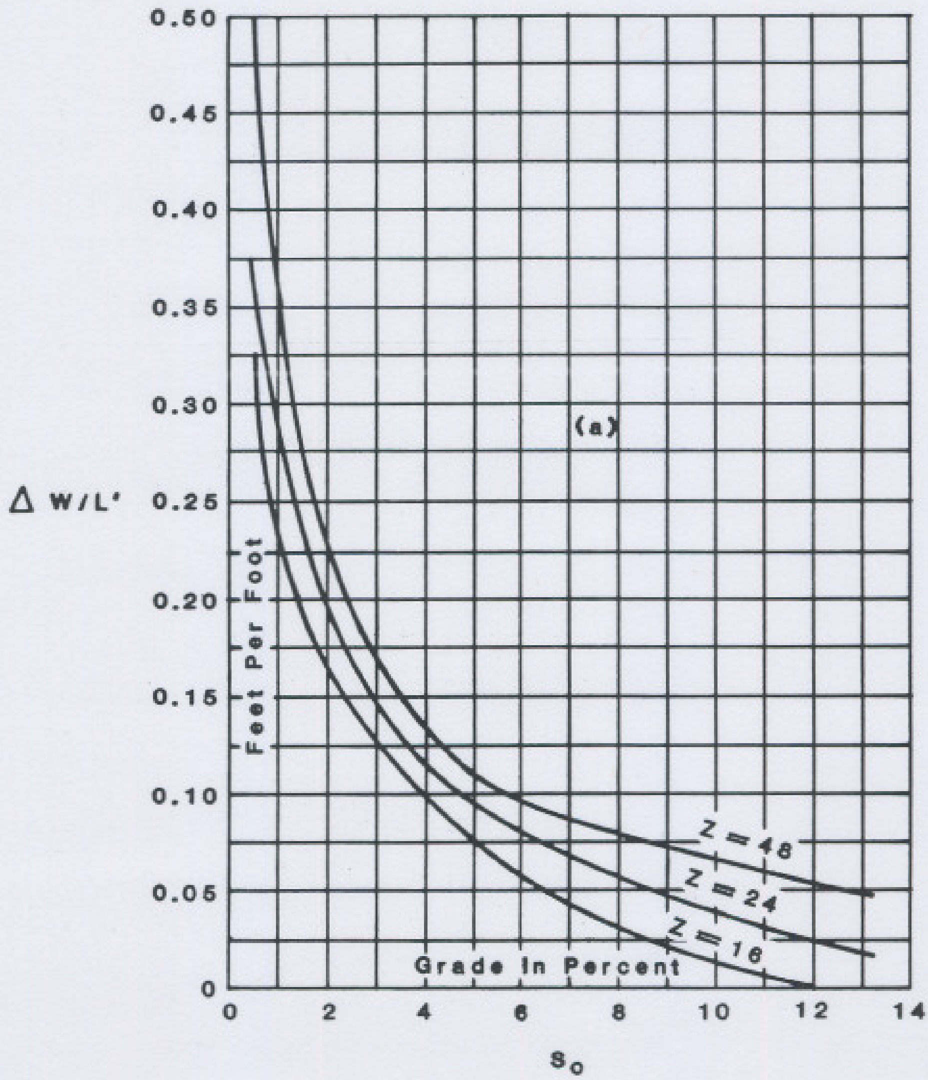
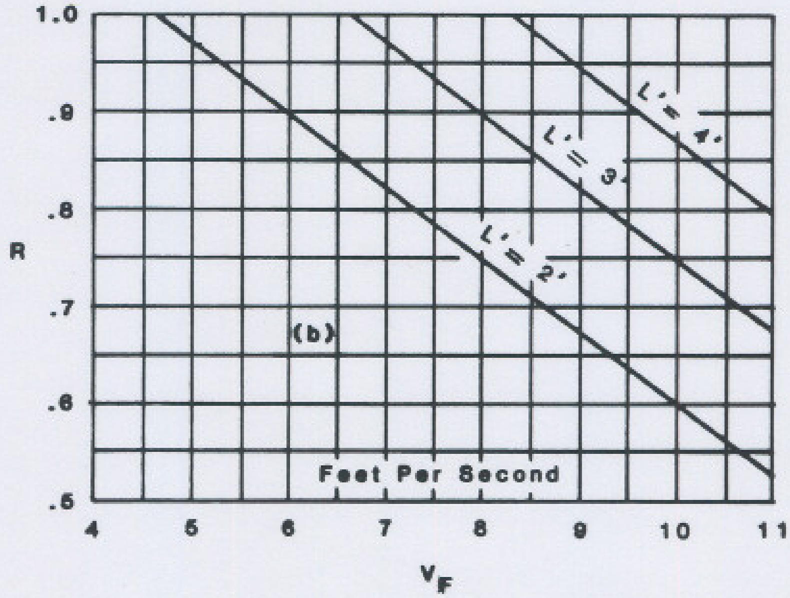


Figure 6-27

GRATE INLET DESIGN CURVES - PWBD GRATES



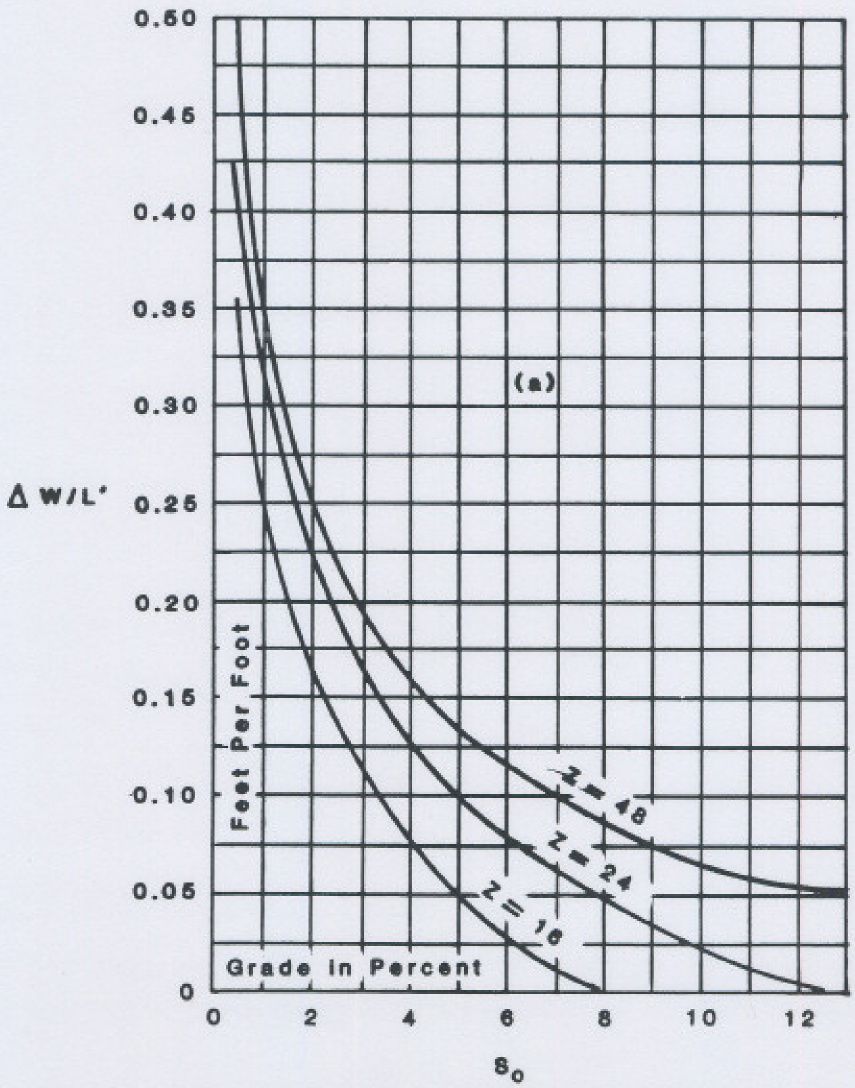
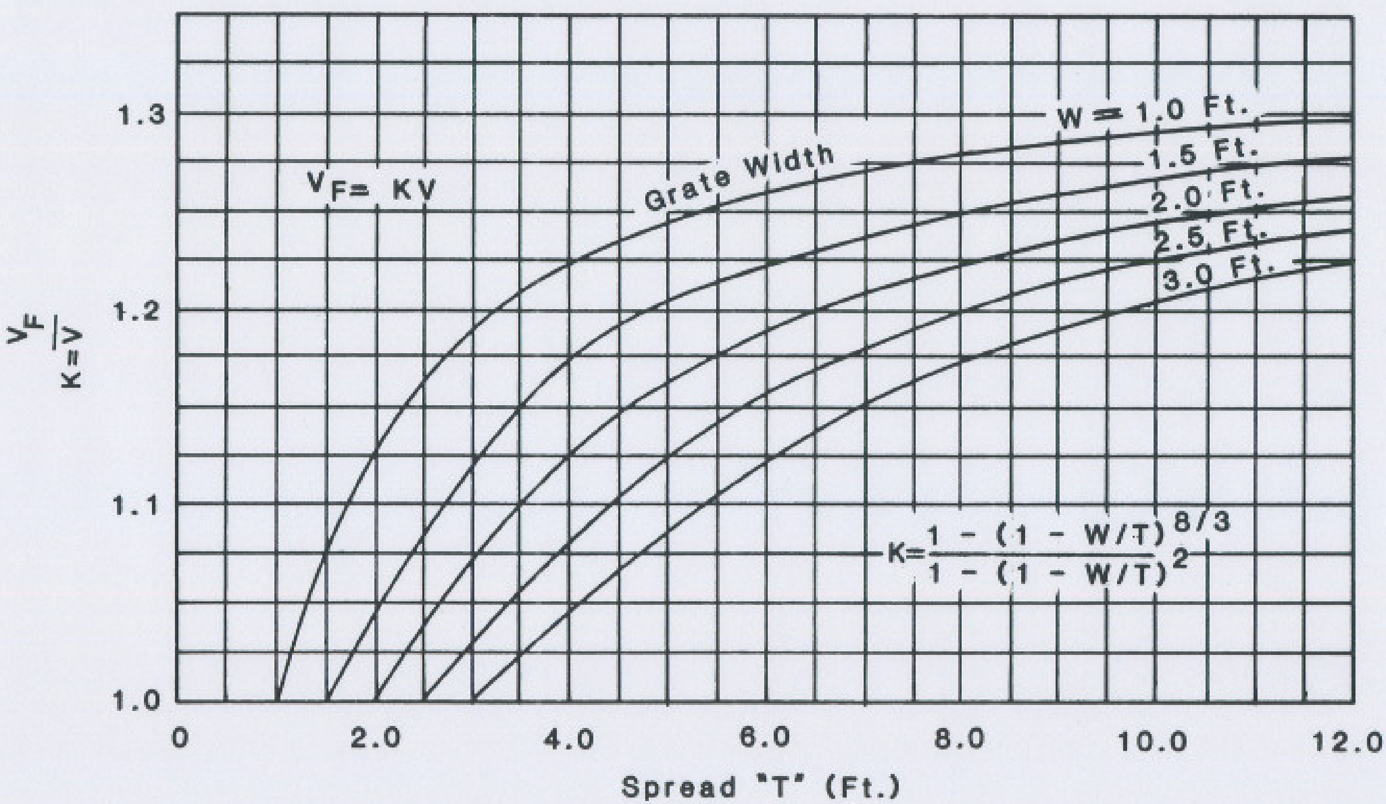


Figure 6-28

FRONTAL FLOW VELOCITY COEFFICIENTS



Spread vs. K (Frontal Flow Velocity Coefficients).

Figure 6-29

PIPE SELECTION CRITERIA

O - ROUND - CORRUGATED STEEL PIPE

Pipe Diameter (In.)	Area (Ft^2)	Minimum Cover (Ft.)	Minimum Metal Thickness (In.)	Minimum Metal Thickness (Gage)	Maximum Fill Heights (Ft.)
2 2/3" x 1/2" Corrugations					
12	0.8				124
15	1.2		0.064	16	100
18	1.8				83
24	3.1				62
30	4.9				62
36	7.1		0.079	14	51
42	9.6				44
48	12.6	1.0	0.019	12	54
54	15.9				46
60	19.6		0.138	10	51
66	23.8				43
72	28.3				44
78	33.2				50
84	38.5		0.168	8	41
90	44.2				33
96	50.3				27
3" x 1" and 5" x 1" Corrugations					
36	7.1				60
42	9.6				51
48	12.6				45
54	15.9		0.079	14	40
60	19.6				36
66	23.8	1.0			33
72	28.3				30
78	33.2				38
84	38.5		0.109	12	35
90	44.2				33
96	50.3				31

Note: The gages shown above are the industry standard. Only the gage shown for each pipe size may be used.

Figure 6-30

PIPE SELECTION CRITERIA

- PIPE ARCH - CORRUGATED STEEL - PLAIN GALVANIZED

Equivalent Round Pipe Diameter (In.)	Span and Rise (In.)	Area (Ft^2)	Minimum Cover (Ft.)	Minimum Metal Thickness (In.)	Minimum Metal Thickness (Gage)	Maximum Fill Heights (Ft.)
2 2/3" x 1/2" Corrugations						
15	17 x 13	1.2				22
18	21 x 15	1.7				21
21	24 x 18	2.3				19
24	28 x 20	2.9		0.079	14	18
30	35 x 24	4.4				18
36	42 x 29	6.5				15
42	49 x 33	8.4	1.0	0.109	12	14
48	57 x 38	11.3		0.138	10	14
54	64 x 43	14.4				14
60	71 x 47	17.4				14
66	77 x 52	21.3		0.168	8	14
72	83 x 57	25.6				15
3" x 1" and 5" x 1" Corrugations						
36	40 x 31	7.0		0.079	14	23
42	46 x 31	9.4				22
54	53 x 41	12.3				21
60	60 x 46	15.6				21
66	66 x 51	19.3	1.0			28
72	73 x 55	23.2		0.109	12	26
78	81 x 59	27.4				24
84	87 x 63	32.1				22
90	95 x 67	37.0				20
96	103 x 71	42.4	1.5			18

Note: The gages shown above are the industry standard. Only the gage shown for each pipe size may be used.

Figure 6-31

PIPE SELECTION CRITERIA

○ - ROUND - CORRUGATED ALUMINUM PIPE

Pipe Diameter (In.)	Area (Ft^2)	Minimum Cover (Ft.)	Minimum Metal Thickness (In.)	Minimum Metal Thickness (Gage)	Maximum Fill Heights (Ft.)
2 2/3" x 1/2" Corrugations					
12	0.8				116
15	1.2		0.075	14	90
18	1.8				75
24	3.1	1.0			56
30	4.8		0.105	12	63
36	7.1				52
42	9.6				58
48	12.6		0.135	10	50
54	15.9				45
60	19.6				49
66	23.8	1.5			44
72	28.3				40
78	33.2		0.164	8	37
84	38.5	2.0			34
90	44.2				N.A.
96	50.3				N.A.
3" x 1" and 5" x 1" Corrugations					
36	7.1				42
42	9.6	1.0	0.075	14	36
48	12.6				31
54	15.9				39
60	19.6		0.105	12	35
66	23.8				32
72	28.3	1.5			38
78	33.2		0.135	10	35
84	38.5				32
90	44.2	2.0			30
96	50.3		0.164	8	34

Note: The gages shown above are the industry standard. Only the gage shown for each pipe size may be used.

Figure 6-32

PIPE SELECTION CRITERIA

⬭ - PIPE ARCH - CORRUGATED ALUMINUM

Equivalent Round Pipe Diameter (In.)	Span and Rise (In.)	Area (Ft^2)	Minimum Cover (Ft.)	Minimum Metal Thickness (In.)	Minimum Metal Thickness (Gage)	Maximum Fill Heights (Ft.)
2 2/3" x 1/2" Corrugations						
15	17 x 13	1.2				15
18	21 x 15	1.7	1.25	0.105	12	15
21	24 x 18	2.3				14
24	28 x 20	2.9		0.134	10	14
30	35 x 24	4.4				13
36	42 x 29	6.5	1.50			13
42	49 x 33	8.4				12
48	57 x 38	11.3				8
54	64 x 43	14.4		0.164	8	7
60	71 x 47	17.4	2.00			7
66	77 x 52	21.3				7
72	83 x 57	25.6				7
3" x 1" and 5" x 1" Corrugations						
36	40 x 31	7.0		0.105	12	29
42	46 x 31	9.4	1.50			25
54	53 x 41	12.3				29
60	60 x 46	15.6				29
66	66 x 51	19.3	1.75			25
72	73 x 55	23.2		0.135	10	22
78	81 x 59	27.4				29
84	87 x 63	32.1	2.00			26
90	95 x 67	37.0				24
96	103 x 71	42.4	2.25			34

<u>Note</u>: The gages shown above are the industry standard. Only the gage shown for each pipe size may be used.

Figure 6-33

PIPE SELECTION CRITERIA

REINFORCED CONCRETE PIPE

Inside Pipe Diameter (In.)	Area (Sq. Ft.)	Maximum Fill Heights (Feet) Above Top of Pipe		
		Class III	Class IV	Class V
12	0.8	8	20	No Limit
15	1.2	9	18	No Limit
18	1.8	10	19	No Limit
21	2.4	11	20	No Limit
24	3.1	12	21	No Limit
27	3.5	12	22	No Limit
30	4.9	12	22	57
33	5.9	13	22	62
36	7.1	11	17	33
42	9.6	11	17	34
48	12.6	12	19	34
54	15.9	12	19	34
60	19.6	12	19	34
66	23.8	12	20	34
72	28.3	13	20	34
78	33.2	13	20	
84	38.5	13	20	
90	44.2	13		
96	50.3	13		
102	56.7	14		
108	63.6	14		

Note: Based on Class C bedding and soil of 130# per cubic foot.

Figure 6-34

CHANNEL LINING CRITERIA

VELOCITY (FT/S)	CHANNEL LINING	COMMENTS
Under 8.0	Grass	Design may require a study of the velocity vs. organic lining charts due to the possibility of eroding the underlying soil.
8.0 - 12.5	Rip-Rap	Maximum permissible velocity depends on the side slope of the proposed channel and size of rip-rap used. For slopes of: 12:1 = 12:5 ft/s, 4:1 = 11.75 ft/s, 3:1 = 11.0 ft/s, and 2:1 = 10.5 ft/s.
Over 12.5	Gabions or Concrete	A great range of velocities may be handled here. All designs lead to a reduction of flow time in the facility and high erosive capability at the outfall.

Note: All drainage facilities shall be evaluated for outfall erosion and service life (flexible vs. nonflexible lining).